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DIVISION OF ENGINEERING AND INDUSTRIAL RESEARCH

NATIONAL RESEARCH COUNCIL

HIGHWAY RESEARCH BOARD

PROCEEDINGS

OF THE

NINETEENTH ANNUAL MEETING

Held at Washington, D. C.

December 5–8, 1939

EDITED BY

ROY W. CRUM

Director, Highway Research Board

WASHINGTON, D. C.

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NATIONAL RESEARCH COUNCIL

The National Research Council is a cooperative organization of the scientific men of America. Its members include, however, not only scientific and technical men but also business men interested in engineering and industry. It was established in 1916 by the National Academy of Sciences.

The charter of the National Academy of Sciences passed by Congress and approved by President Lincoln in 1863 provides that "the Academy shall, whenever called upon by any Department of the Government, investigate, examine, experiment and report upon any subject of science or art." During the World War the Academy offered its services to the Government and at the request of President Wilson created the National Research Council as its active agent to assist the Government in organizing the scientific resources of the country in that time of need.

Later recognizing the potential usefulness of the Council in times of peace as well as those of war, President Wilson, on May 11, 1918, by Executive Order requested the Academy to perpetuate the National Research Council as a permanent organization.

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ORGANIZATION AND MEMBERSHIP

The purpose of the Highway Research Board is to provide a national clearing house for highway research activities and information.

In its practical workings the functions of the Board have been: to provide a forum for the discussion and publication of the results obtained by individual research workers; to organize committees of experts to plan and suggest research work and to study and correlate results; to publish and disseminate information; to provide a research information service; and to carry on fact-finding investigations when special funds are available.

The Board is organized as a project of the Division of Engineering and Industrial Research of the National Research Council. The membership consists of the representatives of 28 technical and commercial associations and organizations of national scope.

Associates of the Board are firms, corporations and individuals who are qualified by interest and activity in highway research and who desire to further the work of the Board by annual contributions to its support.

Much of the technical work is done by committees of specialists and recognized authorities which are organized under seven departments: Administration and Finance, Highway Transportation Economics, Design, Materials and Construction, Maintenance, Traffic and Soils Investigations. The members of these Departments are outstanding men in their fields. Special committees are appointed from time to time to carry on specific projects.

Regular contacts are maintained with all State highway departments, with the engineering colleges and with many city engineering departments, through regularly appointed contact men.

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REPORT OF DEPARTMENT OF HIGHWAY FINANCE

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TRENDS IN HIGHWAY FINANCIAL PRACTICES *

SYNOPSIS

As a guide to highway financial administration, this study of trends provides information concerning possible future income sources and expenditure needs. The discussion includes the trends in gasoline tax rates, registration fees, bonds, tolls, property taxes, Federal aid, diversion, State and local relations, and expenditures for highway transportation services.

It is concluded that gasoline tax rates may be expected to remain approximately stationary, pending the establishment of a definite highway expenditure program. Registration fees, on the other hand, show downward tendencies for passenger cars and an upward trend for trucks. The decreasing use of general obligation bonds is described, together with the growing popularity of highway revenue bonds secured solely by motor vehicle tax earnings, and of public revenue bonds for high-cost structures, secured by toll earnings. The extent of toll financing and its theoretical implications are investigated, revealing a limited place in highway transport of this method of payment. Property tax trends are discussed for both State and local levies, the former being generally reduced to zero, while the degree of local property support varies from zero in some States to considerable amounts in others. On the subject of "diversion," it is noted that although there was growing hostility to this practice in recent legislative sessions, the amount of highway user funds applied to non-highway purposes is expected to reach a new peak for 1939. Changing Federal policies are described, particularly the shift in emphasis to urban problems, and the generally increasing interest in highways from a national viewpoint. Recognition of the cities and local roads in State motor vehicle tax distribution is also described, a survey of State control over the expenditure of these funds revealing that States which have no control exceed those which do, and that many States having legal control find such laws ineffective. Finally, data are presented to show the trend toward increasing expenditures for motoring convenience and safety, and the problem of providing a higher quality of urban highway service is outlined, particularly with reference to the provision of parking facilities.

The report concludes that since many so-called "principles" of highway finance evolve through expediency and legislative fiat, it is important to watch financial trends, and if necessary to challenge them, before they become accepted practice.

Altering financial practices make contemporary highway history a kaleidoscope of ways and means. For since the automobile first revealed itself as a factor in road finance, changes in income sources and expenditure needs have developed more rapidly than statistics to describe them or theories to interpret them. It is an important aid to enlightened highway administration, however, that new developments in financial policy should be anticipated and their probable effects evaluated, in order that actions may be directed with approximate knowledge of future income and future requirements. More fundamentally, since highway financial developments have for the most part evolved through expediency, legislative fiat and the pressure of special interests, financial evolution must be watched, and if necessary challenged, before simple expedients become ruling principles.

The present discussion of trends is primarily a watch, but also a challenge. It attempts to indicate financial princi-

* Prepared for the Department by Wilfred Owen, Research Economist, Highway Research Board.

ples in the making and in the unmaking, to permit better judgment in financial planning and greater scrutiny of financial practices. Source material has not been confined to statistical account of what has already occurred, but has been based primarily upon bills introduced and laws enacted in recent State legislative sessions, as well as upon up-to-date literature, and recent expressions of public opinion.

Trends in highway transportation are often indistinct and seldom universal. Moreover, the fact that they are subject to shift is only proof of the financial mutability which this study of trends has sought to demonstrate. Such disturbing factors, however, are of no serious consequence in the present study, for its primary objective has not been financial prognosis, but rather something in the nature of a moral: that present and future highway policies do not evolve through economic truths or basic rights, but by the normal processes of legislation. Consequently, to the highway administrator, as expert witness, falls the duty of guiding financial policies rather than merely serving as their victim.

TREND IN MOTOR FUEL TAX RATES

Increasing Rates

The multiplication of motor fuel tax income has been the combined effect of increasing numbers of vehicles, greater annual mileages per vehicle and higher tax rates. The question considered here pertains to the tax levy and the extent to which future highway income may be expected to change as a result of altered rates.

Since the first tax on motor fuel became effective in 1919, there have been 124 rate increases in the various States. In 1923, out of 19 States which had previously adopted this method of finance, rate increases were effected in 13. In 1925 the tax was raised by 16 States, in 1927 by 22 States, and in 1929 by 20 States. Altogether the years prior to 1933 account for 110 of the 124 rate increases.

Resistance to Rate Changes

In contrast to this earlier persistence of higher and higher fuel taxes was the tenor of the 1939 State legislatures. These sessions yielded but one rate increase, and this a temporary levy to be effective for two years. At the same time legislative resistance to changing the tax one way or another was demonstrated by the failure of proposed increases in 15 States and the defeat of bills to reduce in at least 10. Since 1932, moreover, out of 330 bills introduced in State legislatures to increase gasoline tax rates, only 14 have passed. The year 1932 was perhaps the beginning of this successful resistance to rate increases, for it was then that a referendum to raise the fuel tax in Maine from 3 to 5 cents was voted down at the polls, while voters in North Dakota for the second time defeated a tax increase, and legislatures in both Massachusetts and New Jersey were likewise unwilling to sponsor higher rates.[1] Recently the people of Missouri voted against an increase in the fuel tax from 2 to 3 cents.

This trend toward stability of rates is demonstrated by a summary of tax increases in each four-year period since 1924. The number of times the tax was raised in the several States during those periods was as follows:

1924-1927	52
1928-1931	37
1932-1935	13
1936-1939	6

Capacity to Pay

That the tendency toward fewer increases in motor fuel tax rates will be

[1] Motor Fuel Taxation in the United States by Finla G. Crawford, 1939.

continued is an opinion not devoid of factual support. For there are several altered circumstances to be considered at the present time which were absent or relatively inoperative during earlier years, and which may readily affect gasoline purchases. In the first place, declining gasoline production costs have ceased to cushion the effect of rising taxes on price, so that motorists can no longer afford indifference to gasoline tax policy. To illustrate, from 1920 to 1931 the price of gasoline without tax fell an average of 16 cents per gallon in 50 representative cities, so that in spite of constant State fuel tax increases, total gasoline prices fell from more than 29 cents per gallon in 1920 to 17 cents in 1931. Since that time, however, gasoline prices have been comparatively stationary or upward, the average 1937 service station price in the same 50 cities being 1½ cents higher than that quoted in 1931, and the price with tax included being 3 cents higher.

The fact that future tax increases are more likely to be reflected in gasoline prices, moreover, is economically of much greater significance today than it would have been in earlier years. The extension of vehicle ownership to lower income groups through easy payment plans and used car bargains has rendered the gasoline consumption bill a primary item in the auto budget. Today the so-called "marginal" group of motorists is much larger and more sensitive to small changes in operating costs, and with total unit transportation costs lower, gasoline payments loom larger as a percentage of the whole.

Willingness to Pay

Not only is the growing importance of paying capacity a deterrent to higher tax rates, but also the factor of willingness to pay. Growing realization that motor vehicle revenues may be spent for general purposes of government has aroused an opposition to motor fuel taxes which the former practice of earmarking such funds for highways had generally obviated. Tardy recognition that some city streets are an integral part of the highway transportation network and as such entitled to consideration in the expenditure of user revenues has also fostered urban hostility to the tax.

In addition to these deterrents to any further general increases in motor fuel taxation, considerable opposition to the tax has been created as a result of pressure brought to bear by organizations interested directly or indirectly in highway transportation. The motorist has been somewhat depressed by the information that he pays in excess of a billion dollars a year in taxes. He is then reminded of the modest size of his weekly salary, with the consequences that distinctions between motordom and martyrdom are rather uncertain. Without commenting on the extent to which such pronouncements may be defended on economic grounds, it is undeniable that future attempts to increase highway revenues by user charges must reckon with this type of opposition.

Fuel Tax Reductions

To speak of motor fuel tax reductions calls for some imagination, for there are few examples of such action and these have involved the reduction of temporary taxes only. It has been noted, however, that in the 1939 legislatures 10 bills for decreasing the motor fuel tax were voted upon, and although they were defeated they are perhaps an indication of what may be expected.

Resistance to gasoline tax reductions is comprised of more than the natural reluctance of legislative bodies to cast aside a profitable source of revenue. Many States, for example, have already assigned a large part of their gasoline tax revenues to the service of State and

local debt. Sixteen States now dedicate more than 20 per cent of such monies to debt payments, and in seven States over one-third of all gas tax receipts go for this purpose. With increasing amounts of the gasoline tax being distributed to local units of government, moreover, it is hardly conceivable that rate reductions could be effected in States where the municipalities are not yet satisfied with the present plan of distribution. The pressure of local units for more State funds may also be expected to continue aggressively in those States which are most dependent on gasoline tax receipts as a result of property tax reductions. Another possible barrier to lower gas tax rates is the growing tendency toward lower registration fees.

Not all the evidence precludes the possibility of gasoline tax reductions, however, for circumstances as well as tax rates differ considerably among the States. Increasing revenues, decreasing debt service and reductions in diversion might provide the setting for lower road user charges on gasoline, particularly in high-tax States.

Temporary Taxes

While resistance to any increase in motor vehicle taxes was evident in the 1939 State legislatures, there were no successful attempts to permit the expiration of temporary increases which had previously been voted for emergency purposes. There are now 12 States which levy temporary motor fuel taxes. The proceeds of these taxes are wholly for non-highway purposes in 4 States, for highway purposes in 5 States, and for both purposes in 3 States. In 6 States the levies expired and were renewed in the recent legislative sessions, one State extending the period for another year and the other 5 for two years. In one State the tax was continued indefinitely by the 1937 legislature, while in another the emergency is expected to pass in 1954. It appears that while only "temporary" increases can be passed under current conditions, the difficulty of getting along without this revenue may prevent a return to the original rate.

Federal Excise Tax

The Federal excise tax on gasoline, enacted as part of the Revenue Act of 1932, was also in the nature of a temporary measure, being adopted for one year instead of the two-year period applying to other levies of the Act. An extra ½ cent tax effective for 6 months in 1933 was permitted to expire, but the 1-cent levy has been maintained. Its latest renewal occurred in the present Congress, the extension being until 1941.

The Interstate Commission on Conflicting Taxation has several times advocated that the Federal tax be discontinued in order that the States might retain this field of taxation exclusively. Recognition of the gasoline tax as a State revenue source for highway purposes was also translated into a resolution by the Council of State Governments, and since the Federal tax was enacted, the legislatures of approximately 25 States have memorialized the Congress to refrain from voting such impositions.

Many persons who have abandoned the hope that the Federal gasoline tax will ever be discontinued have also abandoned the original thesis that the tax is an excise for general purposes of government, and instead have preferred to consider it a permanent source of revenue for Federal highway grants-in-aid. This view has been expressed at the recent Annual Meeting of the American Association of State Highway Officials, which passed the resolution that Congress should be urged to authorize the expenditure of all Federal income from highway users for highway purposes.[2]

[2] A National Tax Association Committee on Highway Finance, Jacob Viner, Chairman,

Other Gasoline Taxes

The Federal government is not the only jurisdiction which has adopted gasoline taxes as a source of revenue. Many counties, and cities in eight States, have levied their own motor fuel taxes. The unproductive nature of these collections and their high administrative cost and nuisance value, however, have discouraged their general application. In the 1937 and 1939 legislatures several counties in Alabama and cities in Florida were empowered to raise revenue in this manner, but apparently there was no further spread of municipal and county taxes. On the other hand, in Missouri, where local gasoline taxes are levied by 60 municipalities, plans are now under way to seek a constitutionl amendment to increase the State gasoline tax from 2 to 3 cents, with the proviso that municipal gas taxes shall be abandoned. This will probably be voted upon at the November elections, 1940. Also in New Orleans the municipal gasoline tax of 2 cents per gallon has recently been repealed by the City Council. In view of such developments, and because of the inherent administrative weaknesses of local motor vehicle taxes, it is unlikely that this method of highway finance will long survive.

Fuel Tax Trend: Conclusion

On the basis of past trends, legislative action and current public opinion, it appears that for the time being the highway administrator can expect few changes in motor fuel tax rates. The stage, it seems, is set for a studied demonstration by the highway departments of the financial needs which the attainment of given highway standards indicate. In the current deadlock between pressure to decrease taxes and pressure to increase them, the establishment of thoughtful guides to the highway expenditure program might well tip the scale for more rational tax rate determination.

advanced the opinion in 1924 that "the Federal government should derive the revenues which it devotes to highways from the special taxation of highway users," except to the extent that such aid may be for national defense or the opening of undeveloped regions. In addition this Committee stated that "the Federal government should not levy taxes on motor vehicles as such in excess of what it spends on highways.—National Tax Association, *17th Proceedings*, 1925, p. 426.

TREND IN REGISTRATION FEES

Past Trend

Methods of highway user revenue collection have undergone a rapid shift in emphasis. In 1926, motor vehicle registration fees produced $100,000,000 more than the motor fuel tax, and constituted 60 per cent of all highway receipts from motor vehicle users. (See Fig. 1 and Table 1.) By 1938, however, these fees amounted to nearly $400,000,000 less than fuel tax receipts, and constituted but a third of total user revenues.

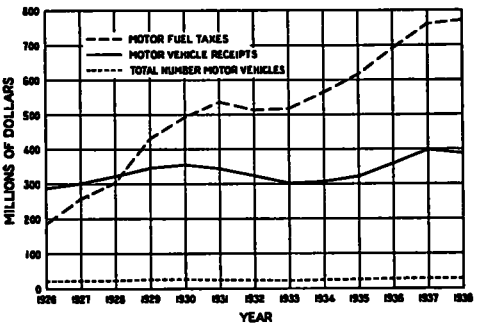


Figure 1. Receipts from State Imposts on Highway Users

The average payment of registration fees per motor vehicle in 1926 was approximately $13. In 1938 this average figure was still the same, in spite of the increase in number of heavy vehicles. It is indicated, therefore, that in general the average rate of passenger car registration fees has been on the downward

trend. It is true, however, that the situation differs in each individual State, with average 1938 registration fee payments for all vehicles being less than $8 in seven States and more than $18 in eight States. Extremes were Georgia's $4.56 average and Vermont's $27.06.

TABLE 1

STATE IMPOSTS ON MOTOR VEHICLE USERS, 1926-1938

Year	Motor vehicle receipts [1]	Motor fuel taxes [2]
1926.....	$288,282,000	$187,603,000
1927.....	301,061,132	258,839,000
1928.....	322,630,025	304,872,000
1929.....	347,843,543	431,312,000
1930.....	355,704,860	493,865,000
1931.....	344,337,654	536,397,000
1932.....	324,273,510	513,047,000
1933.....	301,315,497	518,195,000
1934.....	307,260,000	565,027,000
1935.....	322,974,000	616,851,000
1936.....	359,783,000	691,420,000
1937.....	399,613,000	761,998,000
1938.....	388,825,000	771,764,000

[1] Total receipts, including miscellaneous licenses, fees and fines. Data from Public Roads Administration, Tables MV-2.

[2] Net receipts after deduction of refunds and taxes paid on aviation gasoline. Data from Public Roads Administration, Tables G-1.

Recent Legislation

A review of recent statutes dealing with motor vehicle registration rates is rendered exceedingly difficult by the complexity of registration laws. Changes in the bases of the fees as well as in the rates, and the enactment of special laws for particular classes of vehicles, make it doubtful whether the net result for any special type of vehicle is higher or lower, and whether the general trend is up or down. Study of the laws emerging from the past few State legislative sessions on this subject reveals that a large volume of enactments are unimportant. However, there have been several indications that recent developments have been toward generally lower registration fees.

During 1933, passenger car fee reductions were passed in 8 States and private truck fees were reduced in 5. Legislative action was taken by 33 States in 1933, and the results show 6 States having increases, either in whole or in part, and 16 having decreases.[3] Slight reductions were passed in 1937 for private vehicles in 3 States and for trucks in 2, while major increases in truck rates were enacted in 7 States.

Legislation in 1939 also resulted in many bills of minor importance. However, increases were levied on trucks and busses in Nevada, on busses in Tennessee, and on heavy trucks in Vermont, while Delaware made reductions in passenger car fees, and Idaho and Wyoming lowered the fees on both automobiles and trucks.[4] A Missouri bill proposing a 50 per cent reduction in passenger car rates combined with a one-cent gasoline tax increase was defeated. This attempt, however, as well as the successful reduction of Idaho passenger car fees to a flat $5, coupled with a redistribution of the gas tax, represent a fairly widespread movement toward a low flat fee and increasing dependence upon gas tax revenues.

Deferred Registrations

Probably of greater importance than any other legislative developments concerned with the registration fee has been the enactment of bills deferring the motor vehicle registration date. This practice was first introduced in the State of Maine, which in 1932 changed its registration date from January 1 to March 1 in an effort to encourage all-year use of motor vehicles by assisting the motor-

[3] These figures both include four States which increased fees in some classes and reduced them in others.

[4] Changes made in carrier taxes for regulation rather than revenue have been omitted in this discussion.

ist to keep his car on the road during the winter. For it had been generally observed that the combination of inclement weather and post-holiday pocketbooks exerted strong pressure on the motorist to leave his car in the garage. The resulting idle capacity of transportation facilities meant financial loss to the State not only through unpaid registration fees, but also through consequent reduction in gasoline tax receipts.

The success of the Maine experiment in decreasing unused capacity by recognizing the financial convenience of the motorist was expressed by a 22 per cent increase in gasoline tax receipts for January and February 1932 over 1931. This increase resulted in spite of a 5 per cent reduction in receipts for the year as a whole. Favorable results were experienced in following years, and in 1937 the State attributed the collection of approximately $150,000 additional gasoline tax receipts to the deferred registration plan.

Although winter weather is not so important a factor to the motorist in the southern States, the experience of South Carolina illustrates that considerations of ability to pay are alone sufficient to warrant more careful selection of the time of vehicle registration. In South Carolina the convenience of farmers was served by changing the registration date to October 31, when the sale of crops creates more cash on hand. Since 1933, when the new plan was inaugurated, there has been a marked increase in the number of cars registered during the early months of the year, and gasoline tax receipts have increased approximately 20 per cent.

Previous to the 1939 legislative sessions, 18 States had provided for registration dates other than January 1, most of them April 1. In 1939 further extensions were provided in 7 States, 4 on the first of April, one on the first of March, and another on the first of November, bringing the total to 25 States. Bills were defeated, however, in 5 States and vetoed by the governor in another. In addition to the States with deferred registration dates, 16 have periods of "grace," and one issues half-year plates.

This universal trend toward minimizing the burden of the registration fee, while doubtlessly adding acceptability to that type of charge, at the same time provides evidence of waning enthusiasm for financing highways by means of burdensome lump payments exacted so far in advance of actual road use. If the gasoline tax were collected in the same way, to cover the total charge for a year's highway service, such an outright payment would be quite certain not only to arouse opposition, but to restrict the utilization of highway facilities. For the average annual gasoline tax paid per vehicle is $26 for the country as a whole, and in 8 States amounts to $45 or more.

Registration Fee Trend—Conclusion

It is indicated, therefore, that while a continued trend toward registration fee reductions appears likely for passenger cars, development in deferring or budgeting registration payments may be expected to temper this movement. In the case of trucks and busses, no large-scale reductions are indicated in the near future, and the possibilities of further increases are not remote. It is to be anticipated also that the amount of fees collected will constitute a progressively smaller part of total highway user contributions.

The perplexity arising from analysis of motor vehicle registration laws arouses the hope that less complicated statutes may emerge from future legislation. Multiple classifications of vehicles, diverse bases and combinations of bases, and confusion between cost-of-service and value of service principles yield solu-

tions to the vehicle tax question which often appear more challenging than the problem.

TREND IN BORROWING

The future of borrowing for highways is clouded both by limited data and conflicting trends. On the one hand there are no reliable compilations of State and local highway debt outstanding over a period of years. On the other hand recent disfavor of the general obligation bond for roads has grown simultaneously with enthusiastic endorsement of the highway revenue bond. The future of highway indebtedness, therefore, while a hazardous field for prophecy, is an interesting one for speculation.

A reasonably accurate figure of State highway obligations at the beginning of 1938 places total debt at close to $2,000,000,000.[5] An estimate of local road debt would be well over $1,000,000,000. On the whole, however, it appears that the general highway obligations of both State and local jurisdictions are declining.

General Obligation Bonds

On the subject of State highway general obligation borrowing, it appears that in recent years the pay-as-you-go principle has become fairly well embedded in the financial matrix of a considerable number of States, and that reluctance to repeat large-scale general obligation borrowings such as were customary in the 1920's is fairly well established.[6] This development has been accelerated by the transition from a pioneer period of new highway construction to the less spectacular requirements of maintenance and replacement. In addition, the growth in highway earning power and in Federal highway grants has provided large amounts of current revenues to minimize the necessity for bond issues. On the other hand, it is possible that in some States the realization of highway needs may stimulate a new period of borrowing, and that this might be encouraged by prevailing low interest rates.

In the case of local highway debt, increasing awards of State motor vehicle revenues and Federal funds to local units of government have in many cases provided the financial relief which might otherwise have been sought through additional borrowing and refunding. At the same time tendencies toward State centralization have resulted in the lessening of local financial responsibilities in terms of reduced mileage and State assumption of local debt. Debt limits, tax limits, and the achievement of passable standards on large mileages of local roads have also promoted the continuing downward trend in outstanding local road obligations.

Revenue Bond Development

There are now approximately one billion dollars of public revenue bonds outstanding in the United States, half of these financing domestic service enterprises such as the furnishing of gas, electricity and water, and the other half being for highways, bridges and tunnels. Revenue bonds are issued by public bodies on the security of earnings from the project financed, without backing of the general credit of the State or resort to State taxing power. Practically nonexistent 15 years ago, these bonds have already won a place of significance in public finance. This popular acceptance may be attributed in part to the fact that revenue bond finance recognizes only the users of a service as responsible for its support, thus providing an extension of the benefit principle which has gained acceptance in the financing of highways. Also, since these bonds are issued only

[5] Source: Public Roads Administration. A more complete study is now in process.

[6] State highway debt, which increased 180 per cent from 1923 to 1931, increased only 4 per cent from 1931 to 1936. See Edna Trull, "Borrowing for Highways," Dun and Bradstreet, 1937.

for projects expected to pay their own way, support is given to the careful selection of economically sound projects. Expediency has been a further factor in the resort to revenue bonds, for the absence of any obligation to levy taxes for their redemption and the exclusion of such financing from the public budget have avoided the restrictions of tax and debt limitations.

Revenue bonds for highways issued by the States are secured by a lien on all or part of motor vehicle revenues. The year and amount of some of these revenue bond authorizations are as follows: [7]

Montana, 1931	$4,500,000
Georgia, 1933	26,600,000
Maryland, 1934-1935	5,236,000
Colorado, 1936	25,000,000
Mississippi, 1936	60,000,000

In addition, many States issuing general obligation bonds service them entirely from highway user revenues, so that in practice these issues resemble revenue bonds in servicing methods.

Another form of revenue bond is that issued by a proprietary corporation or authority which issues bonds to the public on the security of tolls. These so-called agency revenue bonds have been used since 1926 [8] for bridges, tunnels, and recently for highways.[9] A few of the most important agencies are the following, which are listed with the amount of bonds issued: [10]

California Toll Bridge Authority	$71,000,000
Delaware River Joint Commission	38,120,000
New York City Parkway Authority	18,000,000
Triborough Bridge Authority	35,000,000
Port of New York Authority	196,834,000
Pennsylvania Turnpike	35,000,000

[7] John F. Fowler, Jr., "Revenue Bonds," 1938, p. 36.

[8] The Port of New York Authority issued bridge revenue bonds in this year for the Staten Island Bridges.

[9] The 166-mile Pennsylvania Turnpike, now under construction.

[10] B. J. VanIngen and Co., "Financing Public Improvements," 1939.

Other agencies in Washington, Kentucky, Delaware, Maryland, Missouri and Illinois have issued approximately $40,000,000 in bonds.

Municipal revenue bonds have been confined for the most part to the provision of domestic services such as water supply and electricity, the possibilities of this method of finance for urban highway facilities having been generally undeveloped. The recent $60,000,000 highway revenue bond issue for express roads in Chicago, however, may mark a new attempt to improve the traffic situation in metropolitan areas. California also made provisions for such undertakings in cities by passage in 1939 of the "Transportation District Act" providing for the organization and operation of transportation districts within two or more municipalities situated in one county, with power to establish comprehensive transportation plans, including freeways and elevated structures, and to designate methods of finance, including tolls.

There are now more than 60 bridges built and operated by agencies representing the public and financed by public revenue bonds. None of these is in default.[11] Revenue bond acts have been passed by nearly all the States, and although many of these are unsatisfactory, the trend toward better enabling legislation may be expected to continue. For "under present conditions it isn't as much a question as to whether the cost of needed public improvements shall be met by revenue bond financing or by general obligation bond financing; it is, rather, a question as to whether the project shall be financed by public revenue bonds or abandoned entirely." [12]

[11] For an excellent treatment of bridge financing, see Robert Klaber, "Bridge Revenue Bonds," 1939.

[12] L. R. Ayers, in "Financing Public Improvements," p. 128. B. J. VanIngen and Company, 1938.

Recent Provisions for Borrowing

In the 1937 and 1939 sessions of the State legislatures, the most important bills dealing with highway borrowing were authorizations for toll bridge and tunnel construction or acquisition, refunding issues, and temporary motor vehicle tax anticipation notes. Provisions for revenue bond financing of toll structures were passed in Alabama, Maine, Maryland, New York, Pennsylvania, New Jersey, Idaho and Indiana.[13] New Mexico authorized $10,000,000 of highway revenue bonds in 1937 and $6,000,000 in 1939. In Illinois, permission granted to Cook County and Chicago makes possible a $60,000,000 bond issue for a system of superhighways, to be repaid from the share of State motor vehicle revenues received by those jurisdictions. A small revenue bond issue was authorized in Idaho, while a projected $35,000,000 revenue anticipation note issue in Oklahoma was declared unconstitutional. The 1939 Arkansas legislature provided for a referendum on the issuance of general highway refunding bonds not to exceed $141,000,000, and Tennessee is also refunding county highway bonds to effect savings in interest. West Virginia in 1937 authorized $10,000,000 of general obligation borrowing, while in New Jersey a proposal for general obligations totaling $60,000,000 has just been abandoned. Maine voters recently defeated a proposed $9,000,000 issue.

Several laws were passed this year restricting borrowing, such as in Iowa and Oregon, or providing for the servicing of outstanding debt by local units with shares of motor vehicle taxes. In fact, legislative action appears to be largely concerned with clearing up errors and omissions in past borrowing provisions, and adopting with caution the newer practice of revenue bond financing.

[13] Some of these revenue bond laws are described in the section of this report dealing with Toll Finance.

Self-Liquidating Projects

In connection with revenue bond trends, recent highway financial developments have attached growing importance to the self-liquidating possibilities of various large-scale construction proposals. A considerable part of recent engineering and financial achievement has resulted from the application of self-liquidating principles as expressed by the revenue bond.

On the subject of self-liquidating projects and spending programs, the testimony of Professor Alvin H. Hansen of Harvard University before the Temporary National Economic Committee last May is of interest. Mentioning the proposed highway spending program and others, Professor Hansen made the following statement:

"Some public investments are self-liquidating in character. These offer no difficulty with respect to financing, and about these there can be, I think, no serious grounds for controversy. Yet, because of an obsolete system of public accounting, we have dumped even these expenditures into our ordinary budget and mistakenly counted ourselves poorer by reason of the public debt incurred to finance these projects."

It was further pointed out in this testimony, however, that there is danger in placing too much stress on self-liquidation, for many undertakings of the greatest social and economic importance, unable to pay their way directly, may be extremely useful and highly productive in an indirect manner.

Right of Way Problem

Further national interest in highway problems which may affect the trend in

highway borrowing is that directed toward land acquisition. The obstacle to highway development created by difficulties incident to land purchase has made it necessary to seek some means of assisting the States and municipalities in financing the acquisition of adequate rights of way for present and future highway needs. In line with recommendations made by the Public Roads Administration this year,[14] bills have been introduced in Congress which would authorize the Public Roads Administration to purchase or condemn land which it would sell back to the States and municipalities for highway purposes.[15] Repayment to the Federal government would then be spread over a long period with Federal funds being realized through the sale of bonds by the Reconstruction Finance Corporation. The latter may also be authorized to make direct loans to the States and municipalities. This proposal will come before the next session of Congress, and if passed will result in a new type of long-term financing in the highway field.

Highway Borrowing Trend—Conclusion

Investigation of the trend in highway borrowing reveals a sharp distinction between that of general obligation bonds, which is down, and that of revenue bonds, which is sharply upward. For although in most States the stage of pioneer highway development has passed, and with it much of the pressure for general credit financing, at the same time the urgency of providing new high-cost facilities for heavy traffic and the demonstrated requirements of metropolitan transportation have created new demands for credit. These factors have evidenced a growing acceptability of public revenue bonds as a method of satisfying present needs, and this acceptability from the viewpoint of highway administration has been complemented by the success of recent bridge and tunnel revenue bond projects in offering attractive opportunities for investment.

In considering the possibilities of bond issues in the future, the fact must be taken into account that many States are still carrying large debt service burdens from past borrowing programs, and that consequently it may not be possible at this time to dedicate larger amounts of highway user revenues to the service of indebtedness (see Fig. 2 and Table 2). From recent legislation it appears that further refunding issues may be expected as a means toward intelligent re-ordering of the present debt structure, and that short-term notes will continue to be issued on the basis of revenue anticipation. The chief development in the future, however, seems to be indicated for revenue bond issues, which, if not abused by overenthusiasm or thwarted by lack of satisfactory enabling legislation, may be expected to assume a leading role in long-term highway finance.

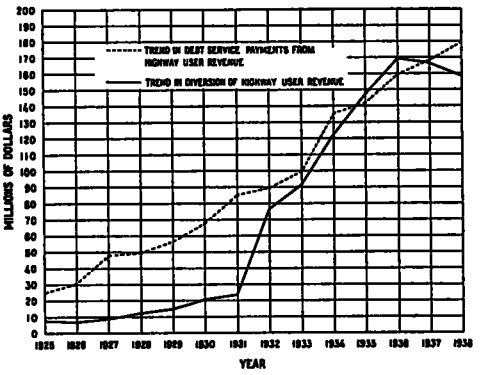


Figure 2. Diversion and Debt Service from Highway User Revenues 1925-1938

[14] Toll Roads and Free Roads, House Document 272, 76th Congress, 1st Session, 1939. p. 114.

[15] Companion bills S-2974 and H. R. 7533 introduced by Senator Hayden and Representative Cartwright.

TABLE 2

TREND IN DEBT SERVICE PAYMENTS FROM HIGHWAY USER REVENUE, 1925-1938

Year	Total	Per cent of user revenue
1925.....	$24,977,000	6.2
1926.....	30,513,000	6.4
1927.....	47,968,000	8.6
1928.....	49,446,000	7.9
1929.....	56,556,000	7.3
1930.....	67,359,000	7.9
1931.....	85,063,000	9.7
1932.....	89,738,000	10.7
1933.....	99,518,000	12.2
1934.....	135,536,000	15.3
1935.....	141,745,000	15.1
1936.....	159,132,000	15.0
1937.....	168,089,000	14.1
1938.....	179,905,000	15.3

TREND IN TOLL FINANCE

The Toll Principle

The use of toll payments to finance special high-cost facilities for highway transportation has received considerable attention during the past few years. Tolls are generally recognized today as being an equitable and expedient method of financing bridges and tunnels when their cost would otherwise prohibit construction. The toll principle, in fact, is embodied in the gasoline tax, which in reality is an adaptation of turnpike finance to "free" road finance. The collection of "tolls" through the State gasoline tax permits payment from large numbers of vehicles without collection difficulties or excessive administrative costs, and accommodates traffic on a system rather than on a single road.

The toll-gate is unsuited for any large part in the financial plan of modern highways. Transportation today must be swift and safe as well as convenient and economical, and while the toll in its proper sphere promotes these desirable qualities, on anything but a limited scale it would tend to interfere with them.

There is, however, a legitimate place for the toll structure in highway transportation in so far as it provides a desired super service. Theoretically, as long as the savings to the motorist by reason of his use of a toll structure are in excess of the toll payment plus motor fuel taxes generated in using the facility, there should be no economic objection to the special charge. No toll facility could be liquidated which did not offset the availability of a competing "free" road by savings in time and operating costs, and by increased convenience. For while the toll structure has often been branded a barrier to free highway traffic, in reality the barrier is not the presence of the toll but the absence of the bridge.[16] "To assert positively that tolls have no place in the American road system would be just as indefensible as to say that tolls should be charged wherever there is lack of money or stress of traffic.[17]

In the use of tolls for financing self-liquidating projects, however, it should be pointed out that the criterion of self-liquidation as applied to the toll structure is not sufficiently inclusive. Considering the highways as a system, the earnings of a bridge consist not only of the revenue collected on the structure but also the motor fuel taxes generated on the whole system by traffic induced by the bridge.

Considerable controversy has arisen with regard to the collection of tolls on the parkways in Westchester County, New York, and on the Merritt Parkway in Connecticut. Thought should also be directed, however, to the circumstances which have given rise to the imposition of these tolls. In Connecticut, the popularity of the Merritt Parkway may constitute evidence of the dangers and in-

[16] D. B. Steinman, American Toll Bridge Association, *Proceedings*, 1935, p. 56.

[17] Editorial, *Engineering News-Record*, October 26, 1939, p. 41.

conveniences of alternate Route 1. On the other hand, the high cost of the Parkway which required resort to the extra toll charge may be a symptom of failure to provide a long-range plan of land acquisition or appropriate financial methods for purchasing expensive rights of way.

In the case of the tolls in Westchester County, an important consideration may be the fact that in New York the use of $73,000,000 of State motor tax payments for general purposes and the denial of appropriate shares of highway aid to heavy-traffic metropolitan areas have created a need for the adoption of additional highway revenue-raising methods. However, Westchester was restrained from collection of the 10 cent toll on the Hutchinson River Parkway by court order in November 1939, on grounds that the Parkway is a link in an important interstate system and hence that local imposition of tolls is illegal. An appeal will be made and a stay of injunction sought. A decision of the New York Supreme Court had previously denied an injunction on the grounds that ". . . . the exaction of a modest fee from those who seek the benefit of a luxurious park system, constructed at enormous expense to the county and its taxpayers, would seem to be eminently reasonable."

Extent of Toll Facilities

The Committee on N.R.A. Code for the toll bridge industry [18] reported in 1933 that there were 312 toll bridges in operation, of which two-thirds were privately owned. Total assets were estimated at $832,000,000 and gross revenues at $59,000,000. The latest survey of toll structures was made by the U. S. Public Roads Administration, which listed 241 toll bridges and 5 tunnels in the United States as of December 31, 1938, more than 100 of these being publicly owned. At that time 6 more bridges were under construction and during the early months of 1939 at least 11 bridges were made free.

Federal Government and Tolls

In recent years the Federal government has been actively interested in the subject of tolls as a means of financing highways and bridges. When the Reconstruction Finance Corporation was created in 1932, authority was granted to provide loans for public and privately-owned toll bridges of a self-liquidating nature. In the following year the Public Works Administration was created and empowered to provide both loans and grants to public bodies for constructing publicly-owned toll bridges. Under these Congressional authorizations $158,000,000 of loans and grants had been made by the early part of 1938, of which $145,000,000 were loans.[19]

Further Federal interest in toll finance was expressed in the Federal Aid Highway Act of 1938, which provided that a report be submitted by the Bureau of Public Roads on the feasibility of certain transcontinental superhighways and of a toll system on such roads. The resulting study led to the conclusion that "since a liberal estimate of revenue for the period 1945-60 is less than 40 per cent of a conservative estimate of debt service, maintenance and operating costs for the same period, a toll system on the roads selected . . . . is not feasible." [20] It was noted, however, that the indicated failure of the system as a whole to be self-liquidating through user tolls did not preclude the possibility of successful toll financing on particular sections of road on the system, although it was estimated

[18] American Toll Bridge Association, *Proceedings*, 1934, p. 13.

[19] American Toll Bridge Association, *Proceedings*, 6th Annual Convention, 1938, p. 83.

[20] Toll Roads and Free Roads, House Document 272, 76th Congress, 1939, p. 86.

that even by 1960 only 300 miles would earn cumulative revenues sufficient to cover costs.

Later in 1939 the Federal government again considered the subject of toll structures in connection with the Administration's proposals for a program of loans for self-liquidating toll roads, bridges, high-speed highways and city by-passes. This spending program, however, was rejected by the Congress.[21]

On the other hand, in keeping with its policy of advocating "free" roads except where special circumstances favor the use of tolls, the Federal government in 1937 passed an act to aid the States in purchasing toll bridges on the Federal aid system for the purpose of making them free. Payment up to 50 per cent of the reasonable worth or construction cost of any toll bridge was authorized out of Federal aid funds. In 1939 the eligibility date under this Act was extended by Congress to July, 1941.

Public Opinion on Tolls

In connection with proposals for the financing of Federal superhighways by tolls, the American Institute of Public Opinion conducted a survey in March, 1938, to determine the reaction of vehicle owners to the payment of extra charges for the use of special high-speed roads. Referring to the longest trip made by each motorist during the previous year, the Gallop poll asked whether a charge of one cent or half a cent per mile would have been paid for making the trip on such "super" roads. The survey answers revealed that 27 per cent of car owners would have paid one cent, and 39 per cent would have paid half a cent.

The fact that these questions had reference to the motorist's longest trip creates speculation as to what answers might have been given with regard to toll bridges, tunnels, and special highway facilities to alleviate metropolitan congestion. However, the significance of the replies seems to lie in the widespread acceptance of the toll principle by motor vehicle owners. Moreover, the largest number of persons interviewed said that safety considerations alone would make them use toll highways, with the next two desirable factors being time saving and relief from strain. It appears to be a reasonable assumption that such considerations would play a large part in the willingness of motorists to pay for better highway service in urban areas if they could get it.

State Toll Legislation

Increasing interest in toll financing has been evident in recent sessions of the State legislatures. In the 1937 sessions Alabama extended the powers of a corporation which it had created in 1935 for the purpose of freeing Alabama highways of toll bridges, the additional authority permitting the issue of bonds for this purpose. Maryland repealed a 1935 Act which had permitted the charging of tolls on certain bridges financed by a 1935 bond issue, at the same time authorizing the State Roads Commission to formulate a comprehensive plan for the construction of bridges and tunnels to be paid for solely through earnings. Georgia authorized local units of government to operate toll structures, while Michigan enacted the provision that all toll facilities should be made free as soon as indebtedness had been paid off. New York State authorized the building of a tunnel under New York Bay from Richmond to Brooklyn, and another underneath Manhattan. In Pennsylvania the Secretary of Highways was granted the power to acquire toll bridges by bond issue, and the State also approved a 166-mile toll road. In New Jersey the Gloucester

[21] Further consideration of self-liquidating projects is contained in the section of this report dealing with Trends in Borrowing.

County Tunnel Act provided for construction of a toll facility under the Delaware River. In general, 1937 toll legislation reveals a recognition of toll payment for financing high-cost structures, and a tendency away from smaller toll facilities which hamper the "free" movement of traffic.

Increasing interest in toll finance was evident to even greater degree in 1939 State legislation. Many States established special commissions to acquire or to operate toll structures. Some of the laws passed in 1939 are the following:

Arkansas: Authorizes the highway commission to maintain a highway toll bridge across rivers or waters bordering the State. Another bill appropriates $120,000 for purchase of privately-owned toll bridges.

Nebraska: Appropriates not more than $63,000 to complete the purchase of any intrastate toll bridges on the Federal or State system.

Idaho: Enacts the "Toll Bridge Acquisition Act" creating a Toll Bridge Committee composed of the governor and 4 members, to acquire toll bridges. Authorizes the issuance of not exceeding $500,000 in treasury notes for this purpose.

Indiana: Creates the Indiana State Toll Bridge Commission to construct and acquire highway bridges over rivers on the State boundaries. The Commission is authorized to issue bridge revenue bonds and collect tolls.

Connecticut: Imposes tolls on the Merritt and Wilbur Cross Parkways, the charge being 10 cents per vehicle or an annual fee of $10 for each Parkway or $15 for both.[22]

Pennsylvania: Authorizes the Department of Highways to lease from the General State Authority any bridge on the State system, to be operated as a toll bridge. Another bill provides for the freeing of all toll bridges acquired by the Department which have cost less than $30,000.

California: Enacts the "Transportation District Act" to permit municipalities to undertake joint plans and finance transportation facilities with tolls.

Michigan: Repeals the Public Acts of 1909 which provided for toll roads.

While 1939 laws in some cases provide for the freeing of toll structures, the extent to which provision is made for the construction and operation of toll bridges emphasizes that such facilities have a recognized place in present-day systems of highway transportation. With growing need for high-cost traffic structures in metropolitan areas and greater resistance to increasing motor vehicle taxes, it is not unlikely that the special toll-financed facility will continue to maintain its modern applicability. For the toll payment today is not simply a means of obtaining money, but a means of getting a job done. If it is used with discretion there can be no logical cry about the demise of the "free" road. For as Mr. H. S. Fairbank has aptly put it, "freedom" of the highway describes "a condition of the use of the public roads, not a condition of cost."[23] Whether we pay for highways directly or indirectly is a matter of financial practice. To go without necessary transport facilities in the name of the "freedom" of the road is no way to arrive at a destination.

Toll Trend—Conclusion

Whereas the toll was once considered a nuisance charge or evidence of monopoly contrary to public interest,

[22] The charging of special license fees for commuters making regular use of toll facilities is an interesting possibility for correcting some of the administrative objections to toll facilities. It is understood that several cities are now considering the adoption of such methods.

[23] "A Highway Program for the United States," presented at the Interstate Conference on Automotive Taxation, October, 1939.

today this method of finance has come to be recognized as an equitable and expedient extension of the benefit principle to the problem of paying for high-cost special service on the highway. As an extraordinary income source its acceptability has paralleled the growing resort to revenue bonds issued for necessary improvements which might otherwise be impossible to attain.

The question of toll popularity in the future depends upon whether the achievement of costly super-service roads and structures demanded by the motor vehicle will continue to remain outside the possibilities of ordinary financial methods. To a certain extent the answer is dependent upon the degree to which other highway financial and administrative problems are successfully settled. For the use of tolls depends in part upon the availability of other revenues and the extent to which the latter are made to yield acceptable standards of highway transportation. But it also appears that the toll is assured a special applicability independent of other financial policies, as an extra charge for service beyond that provided generally from ordinary user revenues.

TREND IN PROPERTY TAX SUPPORT

State Systems

The combination of depressed real estate values, increased motor vehicle tax revenues and concerted effort on the part of property owners has resulted in a widespread decline of the property tax as a source of highway revenue. For the nation as a whole, this downward trend is universal for the State highway systems. State property taxes for roads totaled $34,739,000 in 1921, $10,884,000 in 1932, and in 1938 less than half a million (see Fig. 3 and Table 3).

The tremendous growth in motor vehicle traffic on the State highway systems and the consequent productivity of motor vehicle charges have had much to do with acceptance of the principle that property taxes should no longer be considered a source of primary State highway income. For although the main road may be of considerable benefit to property, its chief function is not land service but traffic service.

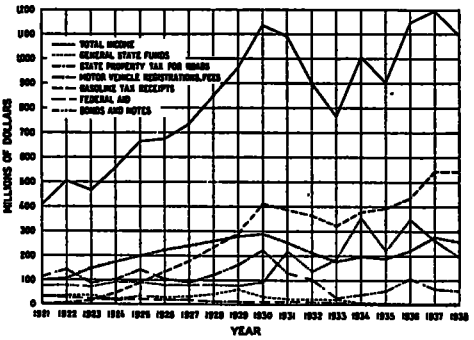


Figure 3. State Highway Income 1921-1938

Secondary and Local Roads

The decline in property tax contributions, however, has not been confined to main roads. In many States property support of other rural roads has also shown startling downward trends. In 1930 the contribution to local roads from property taxes and local appropriations, mostly from property, was close to half a billion dollars, or 60 per cent of all local road income. In the following years of depression, however, the amount of local road property taxes in a number of States was reduced abruptly. For example, in the State of Washington property tax levies for local roads totaled $8,072,000 in 1931 and only $1,396,000 in 1936. California's experience has been similar, with county road tax levies falling from $8,075,000 in 1931 to $1,393,000 in 1937. In 1938, however, receipts increased to $2,451,000. The Michigan experience has been more spectacular than either California or Washington, town-

TABLE 3

STATE HIGHWAY INCOME[1], 1921-38

Year	Total income[2]	General state funds	State property tax for roads	Motor vehicle registrations, fees	Gasoline tax receipts	Federal aid	Bonds and notes
1921	$ 405,973,946	$34,981,680	$34,739,197	$100,232,835	$ 3,358,988	$ 77,456,687	$114,804,202
1922	504,713,429	35,855,278	29,975,825	107,715,649	5,396,677	79,741,492	143,004,141
1923	467,540,849	37,461,579	24,348,478	147,075,966	15,872,884	72,343,401	88,186,784
1924	555,397,045	20,084,324	18,282,878	174,816,973	47,810,615	91,400,832	101,653,327
1925	664,424,571	33,390,642	21,489,004	199,845,163	89,328,340	92,180,406	141,402,022
1926	673,208,430	29,868,048	18,278,994	224,551,631	134,303,154	79,163,201	103,846,301
1927	733,840,751	30,794,645	18,769,561	239,955,128	176,769,657	80,159,671	90,979,230
1928	849,326,598	42,468,386	11,955,782	259,134,820	234,163,826	80,798,365	121,483,599
1929	961,807,038	60,305,631	11,431,349	278,092,734	287,258,416	77,572,691	161,229,297
1930	1,136,673,437	32,136,298	11,181,693	289,801,738	411,109,446	92,462,836	222,288,308
1931	1,092,636,635	22,502,574	12,935,109	253,402,734	386,182,600	218,073,818	130,613,678
1932	898,317,794	20,703,283	10,884,756	211,321,285	363,368,041	136,857,174	104,649,840
1933	764,264,132	20,299,602	7,413,721	176,817,169	321,413,833	185,643,882	27,975,062
1934	1,005,960,000	6,456,000	4,709,000	195,071,000	376,630,000	354,812,000	40,969,000
1935	902,010,000	5,914,000	1,929,000	188,433,000	392,658,000	219,381,000	55,883,000
1936	1,145,590,000	3,756,000	1,377,000	219,088,000	434,876,000	346,281,000	106,235,000
1937	1,195,625,000	2,210,000	775,000	276,840,000	542,813,000	262,762,000	65,009,000
1938	1,096,908,000	5,473,000	489,000	256,817,000	542,638,000	196,826,000	57,505,000

[1] Data compiled by the Public Roads Administration, Tables F-1 (1921-33) and SF-1 (1934-38).

[2] This column includes certain miscellaneous current revenues and transfers from local units not listed in the other columns.

ship and county property taxes having declined from $24,430,000 in 1929 to $1,070,000 in 1937 (see Fig. 4 and Tables 4a, 4b, 4c). Reduction in Wisconsin has been from $18,580,000 in 1930 to $4,493,000 in 1935.

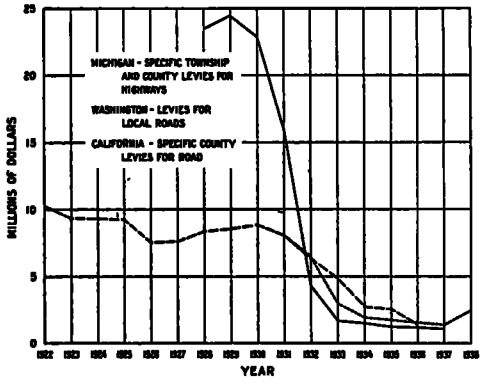


Figure 4. Local Property Taxes for Rural Roads—(Michigan, solid line; Washington, broken line; California, dotted line).

TABLE 4a

PROPERTY TAX TREND—CALIFORNIA [1]

Specific County Tax Levies for Road Construction and Maintenance

Year	Number of counties making levies	Total levy
1931	56	$8,075,000
1932	55	6,455,000
1933	47	2,975,000
1934	37	1,910,000
1935	35	1,729,000
1936	34	1,559,000
1937	34	1,393,000
1938	38	2,451,000

[1] "Highway Facts," California State-Wide Planning Survey, 1939, p. 38.

Where motor vehicle revenues are of considerable magnitude, where the State has either assumed control of large rural road mileages or adopted a generous subvention policy, and where highway facilities are relatively adequate, conditions have been favorable to the passing of the property tax as a source of rural road income. Such has been the case in North Carolina, Virginia, West Virginia and Delaware, in which States centralization has been a prime factor. It is also apt to be true of States where all motor vehicle taxes are used for highways.

TABLE 4b

PROPERTY TAX TREND—MICHIGAN [1]

Specific Township and County Property Tax Levies for Highways

Year	Total levy	Per cent of 1928
1928	$23,480,000	100.0
1929	24,430,000	104.0
1930	22,860,000	98.4
1931	15,920,000	68.0
1932	4,220,000	18.0
1933	1,680,000	7.2
1934	1,490,000	6.3
1935	1,210,000	5.2
1936	1,180,000	5.0
1937	1,070,000	5.0

[1] Michigan Highway Planning Survey

TABLE 4c

PROPERTY TAX TREND—WASHINGTON [1]

Property Tax Levies for Local Roads

1922	$10,281,000
1923	9,376,000
1924	9,323,000
1925	9,263,000
1926	7,556,000
1927	7,603,000
1928	8,396,000
1929	8,536,000
1930	8,852,000
1931	8,072,000
1932	6,366,000
1933	4,872,000
1934	2,702,000
1935	2,557,000
1936	1,396,000

[1] State of Washington, Highway Cost Commission, Supplemental Report, 1937, p. 108.

Property taxes, however, are still an essential item in the highway budgets of a large number of States. In Iowa, 1935 highway property taxes of $9,800,000 exceeded gasoline tax receipts and constituted more than a third of net direct expenditures for roads and streets. Kansas property taxes amounted to

$7,200,000, while motor vehicle taxes contributed $11,600,000. In Missouri, likewise, nearly $8,000,000 from property were but two million less than collections from the gasoline tax.

Trend in Nine States

Surveys of highway finance in 1930-1932 conducted by nine States in cooperation with the Bureau of Public Roads provide evidence of the extent to which property levies were then relied upon in those States.[24] Data summarized in the 1936 Report of the Department of Finance[25] revealed the following amounts of property taxes available for rural highways:

System	Property taxes	Percent of total direct expenditures on systems
State highways..	$ 9,829,000	7
County roads....	69,654,000	70
Township roads..	48,234,000	89
All rural roads...	$127,717,000	42

That this contribution of approximately an eighth of a billion dollars far exceeds the property contributions being made for rural roads in these nine States today is evident even from the limited data available. In the case of the State highway systems, with the exception of $330,000 collected by Colorado, there were no property taxes being used for highways in 1938. In addition, the amount of motor vehicle revenues distributed to local roads and streets in these States during 1938 was three times as large as in the years of the surveys, indicating considerable relief to property. The downward trend in property contributions has already been noted for Michigan and Wisconsin, while the consolidation of 6,000 additional miles of rural roads under State control in Minnesota and New Mexico may also indicate a shift from local to State financing sources. This evidence, together with the knowledge that property support has been reduced by depression and supplemented by Federal aid leads to the conclusion that property contributions are much smaller in these States today than in 1930 and 1932.

City Street Support

From these financial surveys, property taxes spent for city streets in the nine States studied were found to equal $156,000,000 or 84 per cent of total direct street expenditures of $186,000,000. During depression, however, the pressure of relief requirements on municipal budgets, combined with shrinking municipal revenues, at first resulted not only in virtual cessation of capital outlays, but also in the deferment of a large part of necessary maintenance. Later, as locally raised revenues failed by increasing margins to accommodate the needs of the cities, Public Works, Works Program and regular Federal Aid Funds, together with increasing State allotments of motor vehicle revenues, alleviated the financial distress. A considerable amount of work relief expenditure was made on streets in municipalities during this period, since the unemployment problem was most acute in urban areas, and street work could readily be provided.

That the trend in property tax support of city streets has been downward, in many cases to zero, is well established by scattered data on declining expenditures for city streets and increasing State and Federal assistance. As isolated examples, property taxes for city streets declined in Wisconsin from $3,114,000 in 1930 to $996,000 in 1935, while in Michi-

[24] The States are Colorado, Illinois, Michigan, Minnesota, New Hampshire, New Mexico, New York, Wisconsin and Wyoming.

[25] Summary of Financial Surveys in Nine States, *Proceedings*, Highway Research Board, Vol. 16, 1936.

gan city street capital outlay and maintenance expenditure from State and local sources was reduced almost one-third from 1930 to 1935. This reduction, moreover, took place in spite of the fact that the cities received some $3,500,000 of motor vehicle revenue in 1935 and none in 1930.

Further evidence of financial trends for city streets is furnished by a recent compilation of methods used for street support in 400 cities throughout the country.[26] Of these cities, 136 reported that 100 per cent of street costs were ordinarily paid by special assessments against benefited property. Among sources of revenue other than special assessments or general funds, 79 cities mentioned P.W.A., W.P.A., or State gasoline tax monies, and some cities were financing all street work from these sources. Moreover, a number of municipalities stated that no capital improvements had been made during the past few years, or that taxes were being levied for maintenance only.

Property Tax Trend—Conclusion

In many States acceptance of the principle that property should contribute to the financing of roads from which it benefits has met with repudiation. Where the responsibility for road support has not yet been shifted from property to motor vehicles and the Federal government, efforts are tending in that direction. The experience in a number of States where property is no longer a factor in road finance, however, indicates that in other States attention should immediately be given to means of determining equitable shares of the road bill payable by vehicles and by real estate before the pressure to relieve property of what may be a fair charge against it has deprived the highway program of this revenue source.

[26] "Financing City Pavements," *American City*, August 1937, pp. 83-89.

TREND IN FEDERAL AID

Change in Concepts

Federal participation in highway construction was formerly confined to rural mileage on the Federal-aid system, which originally comprised up to 7 per cent of the rural road mileage within each State. The expenditure of these funds within municipalities exceeding 2,500 population was specifically prohibited in order that the more pressing needs of intercity rural routes would be given prior attention. In the years 1919 through 1930, therefore, aid from the Federal government to the State highway departments constituted a rural road program, with average authorizations of $75,000,000 per year.

In 1931, however, confronted with the problem of unemployment and general economic stagnation, the Federal government found that emphasis in highway policies had suddenly shifted from providing a national system of rural roads to creating employment and stimulating the construction industry. Regular Federal aid authorizations were raised to $125,000,000 per year, and additional emergency grants from 1930 through 1935 totaled $1,200,000,000. The effect of these new conceptions was to increase the income of State highway departments by as much as $354,000,000 of Federal money in 1934, and to render Federal sources of State highway revenue from 1931 to 1938 larger by well over $100,000,000 than contributions of motor vehicle registration fees during the same period. In addition, considerable sums of Federal support were received directly by local units of government.

Changes in Federal policy brought about by depression, however, were not confined to objectives or amounts of expenditure. For as a result of the availability of large roadbuilding appropriations, and in keeping with the purpose of relieving unemployment, federal interest in highway building was broad-

ened to include city streets and secondary roads. What had been taking place gradually in the evolution of State aid to local units of government now began quite abruptly in the case of Federal Aid. By the end of October 1939, municipal highway and grade crossing projects completed or approved for construction with Federal funds authorized since March 1, 1933 totaled $393,000,000. With resumption of regular Federal aid in the fiscal year 1936, it was provided that these funds be made available for expenditure on extensions of the Federal aid system into and through municipalities.

Secondary Road Program

Beginning with the fiscal year 1938, the improvement of secondary roads with Federal funds, first undertaken as an emergency measure, became a part of the regular Federal aid program. The new policy, which authorized $25,000,000 for each of the fiscal years 1938 and 1939, differs from that of the emergency in that the States must match Federal funds and select an eligible system of secondary roads not exceeding 10 per cent of total rural road mileage. Similar procedure had met with such success in developing the primary Federal aid system that its extension to secondary roads has been widely endorsed both for financial and administrative reasons. The fact that there are approximately 1,000,000 miles of important secondary roads, including State aid highways and principal county trunk highways, and that only 37 per cent of these have been improved to gravel surface standards, suggests that considerable further attention must be given to the secondary road problem.[27] The determination of secondary systems now being carried out by the State highway departments may be expected to encourage this trend by assuring the expenditure of federal funds where future traffic and land use considerations warrant.

New Financial Needs

In addition to the requirements of the primary and secondary Federal aid systems, as well as the grade crossing elimination program, two new developments have recently been given study by the Federal government. One of these is a system of interregional highways which would join the larger cities of all the States for the principal purpose of serving relatively long-range highway transportation. On these roads attention would be directed to safety, time saving and convenience of travel by limitation of cross traffic and local access roads, by protection of roadside land uses and through the by-passing of small towns. These express roads would follow existing primary routes where feasible, but would depart from them where routing of traffic or acquisition of sufficient rights of way required.

Added to the regular assistance now being granted to cities for street improvements, increasing attention is also being directed toward a program of expressways through and within the larger cities to alleviate the congestion which discourages motor vehicle use in urban areas. The magnitude of this problem is generally familiar, not only to half the population of the United States which lives in the metropolitan areas of Class I municipalities, but also to those of the other half who have penetrated the American City by automobile. The disposition of newly introduced Federal legislation to assist the States in right of way acquisition will determine in the next session of Congress whether a beginning can be made toward freeing the flow of traffic through urban bottlenecks.[28]

[27] Governor H. H. Blood, *American Highways*, Oct., 1939, p. 42.

[28] Proposed Federal legislation for acquiring rights of way is discussed in the section

Federal Expenditure Trend—Conclusion

Federal participation in future road-building programs is dependent not only upon demonstrated highway needs, but upon government policy with regard to public spending, which in turn is dependent upon general economic conditions. Highways provide excellent channels both for work relief projects and compensatory spending. Growing acceptance of the theory that an outlet for savings must be provided in the form of investment expansion to prevent chronic depression may in the future lead to new capital outlays for roads by the Federal government. On the other hand, business recovery and the probability of government economies may reduce Federal support.

The hazard of venturing opinion upon future sources of highway revenue, therefore, is nowhere more evident than in the field of Federal participation. This is also true because policies of the Federal government are dependent upon circumstances of national scope, and its highway policies must be formulated with reference to numerous counteracting and modifying factors. Yet the conclusion appears inescapable that interest in the development of a modern highway system has at no time been more convincingly indicated on a national basis.

TREND IN DIVERSION

Considerable legislative activity has been directed in recent years toward the so-called diversion of highway revenues to non-highway purposes. With only one State diverting in 1921, and only 7 in 1929, depression and the consequent dilemma of obtaining public revenues anywhere and quickly found law-makers turning more frequently to the earning capacity of the highways. In 1938 they were obtaining $158,000,000 in 37 States. It should be noted, however, that in 8 States diversion was a negligible amount, while in 14 others it was less than 10 per cent. On the other hand 6 States were diverting more than 25 per cent of all highway income.

When in 1936 the peak diversion of $169,000,000 had been reached (see Fig. 2 and Table 5), comprising 16 per cent of all motor vehicle tax revenue, a counter-attack was already being launched by those who would preserve the motorists' contributions for motor road purposes. To date, seven States [29] have passed anti-diversion amendments, while in the 1939 legislative sessions five States [30] took action toward that end, and actual diversions were defeated in Tennessee, Florida and Washington. In addition North Dakota reenacted its present anti-diversion statute.

TABLE 5

TREND IN DIVERSION OF HIGHWAY USER REVENUE, 1925-1938

Year	Total	Per cent of user revenue
1925.....	$7,179,000	1.8
1926.....	6,903,000	1.5
1927.....	8,793,000	1.6
1928.....	12,046,000	1.9
1929.....	14,697,000	1.9
1930.....	20,160,000	2.4
1931.....	23,600,000	2.7
1932.....	76,747,000	9.2
1933.....	91,577,000	11.2
1934.....	122,150,000	13.8
1935.....	147,143,000	15.7
1936.....	169,344,000	16.0
1937.....	161,413,000	13.5
1938.....	158,284,000	13.5

on Borrowing. Of interest in this connection is a law passed by California in 1939 providing for acquisition, including acquisition by condemnation, of real property or interests therein for State highway purposes, including excess acquisition for future needs, and for the management, control, leasing, sale or exchange of such property of interests.

[29] Kansas, Colorado, Missouri, Minnesota, California, Michigan, New Hampshire.

[30] Idaho, Iowa, South Dakota, Wisconsin, Nevada.

On the other hand 1939 progress in diversion was impressive. Resolutions to place anti-diversion amendments before the electorate failed or were not acted upon in 11 States, while an anti-diversion statute was voted out in Indiana. Moreover, the size of appropriations from the highway fund to general purposes in a number of States indicates that the total for this year may exceed the 1936 peak.[31] In New York the general fund will receive $73,000,000 from the motorist, Rhode Island will continue using one-third of the gasoline tax for relief, and Nebraska increased appropriations to the State assistance fund from 10 to 20 per cent of the gasoline tax. Pennsylvania appropriated $11,570,000 from the motor license fund for servicing the State debt. Ohio permits counties to use 25 per cent of gas tax revenues prior to April 15, 1941, for poor relief, provided reimbursement is made for such borrowings before May 31, 1941. But previous experience with temporary borrowing from the highway fund permits no assurance that such loans will be repaid.[32] In the case of California, however, it has recently been ruled that temporary transfer of motor vehicle revenue to the general fund is contrary to the anti-diversion constitutional amendment adopted last year.[33]

The successful passage of anti-diversion amendments appears to have established the fact that public opinion favors the use of special motor vehicle revenues for the development and maintenance of highways. A poll at the New York automobile show this year on the question "Should all gasoline taxes and registration fees be used for highway purposes?" yielded similar indications, with the vote 8,551 yes and 968 no.

It would be naive to suppose, however, that after passage of a constitutional amendment a desirable standard of highway service is thereby assured. Other provisions, in the form of adequate administration and planning, are necessary to guarantee how, where and to what advantage highway money shall be spent. For example, investigation of administrative control over the expenditure of State-wide money has revealed the interesting information that in the majority of cases where there is constitutional provision barring diversion, the State has no control over how motor vehicle revenues distributed to local governments shall be spent. In addition, the difficulty of framing an amendment which will endow the term "highway purposes" with sufficient breadth or limitation to fit particular circumstances remains a serious problem.

TREND IN STATE-LOCAL RELATIONS

Highway transportation comprises travel over an integrated system of roads and streets. Motorway finance, however, was originally conceived on a different pattern, consisting of the concentration of motor vehicle revenue expenditure on the main lines and the abandonment of other road and street problems to the financial solutions of local jurisdictions. This financial pattern was a wise expedient when the priority problem consisted of building main lines. Today, however, with many parts of the highway system having legitimate claim on State revenues, the highway administrator must think in terms of an integrated network.

Figures of motor vehicle tax allocations to local units of government are evidence that considerable financial recognition is now being given to the

[31] A $21,000,000 relief bond issue recently voted in New Jersey is expected temporarily to ease the pressure for highway fund diversions in that State.

[32] One example of repayment is that by the Georgia legislature in 1937, which provided for the restoring of 1935-36 diversions.

[33] An opposite situation has been permitted by Connecticut; which in 1937 authorized the highway department to borrow from the general fund for highway purposes at any time, for a period not exceeding three months.

systematic nature of highway traffic. In 1937 $277,000,000 was distributed by the States to lesser units of government, this amount constituting the most generous assistance yet awarded for local roads and streets. In 1938 the distribution of $273,000,000 was approximately as great. But such figures of tax allocations are by no means the only necessary criteria in judging the extent to which financial aid to local units is in the interest of the motorist. Equally important are the questions of where and how well the money is spent.

Laws Affecting Local Roads and Streets

In the 1939 legislatures 122 bills were introduced on the subject of allocating motor vehicle revenues, and laws were enacted in nine States. In addition, numerous bills were passed concerning other financial and administrative matters of concern to the local units. From this legislation and the laws passed in 1937, there is noted not only an increasing recognition of financial claims made by counties and municipalities, particularly the latter, but in addition a growing concern for the administrative task of redesigning highway financial policies on a system basis.

In 1937 Indiana passed its Motor Vehicle Highway Account Act which required budgeting of State funds distributed to counties and cities, and imposed limitations on the amount of such monies which might be used for maintenance. Minnesota amended its statutes relating to the use of gasoline taxes by the counties with a provision that 50 per cent of this money should be apportioned to the towns. In Michigan the registration law was amended to permit the use of registration fees for maintenance of city and village streets.

During 1939, problems of city street financing continued to be the subject of considerable legislative activity in the field of State-local relations. In South Dakota, Wisconsin, Minnesota and Arkansas attention was directed to the inclusion of city trunkline extensions as part of the State highway systems. In Wisconsin both the State and the counties were made responsible for construction and maintenance of city and village streets serving as connecting routes for State and county systems. The highway commissioner of Minnesota was authorized to enter into contracts with municipalities for the construction, improvement and maintenance by municipalities of streets forming part of the trunk highway system, the law having previously applied only to cities with over 50,000 population. The Montana legislature also passed a bill which stipulated that counties may use their roadbuilding machinery on the streets of cities and villages of less than 4,000 population.

Other 1939 State laws affecting local finance were Oklahoma's distribution of 15 per cent of registration fee receipts and 5 per cent of gas tax revenues for streets in cities and villages, increased aid of approximately 50 per cent to cities in Washington, the allotment of $8,500,000 in gasoline taxes to second-class township roads in Pennsylvania, an appropriation of 20 per cent of the motor fuel tax to Idaho counties to compensate for a reduced registration fee,[34] and the stipulation that after June, 1941, Nebraska counties will receive three-eighths of the gas tax receipts rather than three-tenths. Massachusetts appropriated $9,600,000 annually for 1940 and 1941 to cities and towns for highway work, while annual allotments of $800,000 were made to Maine cities for 1940 and 1941 and $2,750,000 to State-aid and town roads in Vermont. Further allocations of State revenue to local units were defeated in Arizona, Arkansas and Oregon.

[34] The Constitutionality of this law has been challenged.

Centralization

Although the trend toward road consolidations and centralized administration has not continued on the scale reached in depression years, several developments occurred in 1939 legislation. In North Dakota the legislature approved a measure to be submitted to the voters in 1940 which would authorize the setting up of procedures for the abolition or consolidation of counties. In Arkansas road districts were abolished, and the administration of local roads was transferred to the county highway commissions. Provision was made in Idaho to include all rural roads in a State-wide system, counties and road districts retaining administrative powers but acting as agents of the State, while in South Carolina the addition of 1,000 miles to the State system was authorized during each of the years 1939 and 1940. Trends toward consolidation and centralization took the opposite direction in Maryland, however, where until this year the roads in all but three counties had been maintained by the State under an optional agreement provided in 1933. Half a dozen counties have now chosen to maintain their own roads.

State Control Over Local Expenditures

Judgment concerning the adequacy of State-aid programs for roads and streets is not so much a matter of the amount of money available as of the degree of State control exercised for its proper expenditure. Although increasing attention has recently been given to means of providing State approval and supervision over local projects financed with State funds, current progress leads more to the realization of things undone than of any far-reaching achievement.

The following information concerning the degree of State jurisdiction over motor vehicle funds distributed to local units of government has in most cases been abstracted from statements supplied by the State highway departments.

ALABAMA: According to law it is the duty of the State Highway Department to establish standards for maintenance and construction work performed with State aid, and to provide information or assistance when called upon by local officials. The State also is directed to determine the character and have general supervision over the construction and maintenance of all highway work involving State funds.

ARIZONA: The State Highway Department has no lawful power to control the expenditure of motor vehicle revenue distributed to local units of government, and cannot require that the money distributed to the counties be spent for highway purposes.

ARKANSAS: Statutes authorizing the return of State motor vehicle imposts to the counties require that such funds be spent for highway purposes, but the highway commission has no authority to pass upon local projects or to supervise the work.

CALIFORNIA: Statutes require that the portions of motor vehicle fees and gas tax receipts distributed to the counties be used for road purposes. The State Division of Highways is not given authority for supervision or inspection of the work performed by the counties with these funds, but the law does provide that the counties submit an accounting of expenditures to the Division of Highways in order to show that the funds were expended for road purposes. The auditing of these statements is the extent of supervision by the Highway Division. In the expenditure of gas tax allotments to the cities, expenditures are under immediate supervision of the State, the set-up being similar to the relationship between the Public Roads Administration and the States in the expenditure of Federal Aid. The cities are required to submit to the State for approval project statements of work proposed during each year. In the case of allocations for use on State highway routes through cities, the Division of Highways plays an active part in the selection of projects, and in many cases the work is performed by the State.

COLORADO: The statutes provide that State motor vehicle funds received by the counties shall be spent on public highways, but the State Highway Department has no jurisdiction in the matter except that one of the functions

of the State highway engineer is to "furnish plans for all bridge construction of 20 feet or more span on county highways; and also plans for county highway construction when requested." In addition it is provided that the State engineer shall "give county boards and other officials charged with highway work such information and advice as he may have at hand and as may be requested."

CONNECTICUT: The expenditure of all State motor vehicle funds distributed to local units of government is under control of the State Highway Department. The highway commissioner is directed to make agreements with town officials relative to the location, construction, improvement or maintenance of highways with such money.

DELAWARE: All highways are under State control.

FLORIDA: Gasoline tax receipts distributed to the counties must first be applied to annual road bond requirements, but a specified part of any surplus remaining may be used for road construction and maintenance, projects being solely at the discretion of the county commissioners.

GEORGIA: All revenue from motor vehicle license tags is allocated to the Post Roads Division of the State Highway Department, which allocates these funds to the various counties and supervises the work of construction. Such funds cannot be used for other than highway purposes. This system is found to be very effective.

IDAHO: The 1939 act of the Legislature appropriating 20 per cent of motor fuel tax revenues and not less than $1,000,000 annually to the counties provides that such money shall be used first for highway bond interest and sinking fund requirements, and the balance for current highway expenditures. Under this act all of the highways in Idaho are included in a State-wide system, with local units of government acting as agents of the State. The Department of Public Works has set up a uniform system of accounting, as empowered, and will make periodic inspections of the books of local units to ascertain whether State money is in fact being spent for the purposes stated. It is doubtful whether the Department has "legal power to require that such money be spent on roads and streets" without resort to court action. How far the Department is authorized to control these expenditures may be determined hereafter by judicial interpretation or future legislation. The local units have thus far manifested a willingness to cooperate with the Department of Public Works and have adopted its forms for a uniform accounting system. They are conscious of the conditions imposed by the Hayden-Cartwright Act, and seem willing to conform to the requirements and suggestions of the Department relative to disbursement of funds allocated to them.

ILLINOIS: The Department maintains full control over expenditure of motor fuel tax funds distributed to local units of government. Requests are made to the Department by counties or municipalities for the improvement of certain highways and after such requests are approved, local units are notified of such approval, after which proposals on the work are requested. No contract is awarded without the approval of the Department. As soon as a contract has been awarded allotments are made through the State Department of Finance for the amount provided. The Department maintains a system of auditing of expenditures made in connection with all contracts in order to see that all moneys are properly expended. General supervision is exercised over construction work as it progresses.

INDIANA: Counties may use State motor vehicle funds for highway purposes only. They also are required to budget these monies, and the budget must be approved by the State Board of Accounts as well as the State Tax Board. This gives absolute control over the expenditures. When this Act went into effect in 1937, the highway commission immediately appointed a County Contact Engineer who exercises the supervision fixed by statute.

IOWA: The Highway Commission does not have direct supervision of the expenditure of motor fuel tax funds which go to the counties for secondary roads, but it does have general supervisory control. Any such funds used for construction must be spent on secondary road programs prepared by the county board of supervisors and approved by the State Highway Commission. Detailed plans for such construction work must conform to standard specifications of the Commission. If any bridge built from such State funds costs more than $2,000, the contract awarded by the county board of supervisors is subject to approval by the State Highway Commission before becoming effective. Likewise any construction contract aggregating more than $5,000 is subject to the commission's approval. There are many

ways of avoiding and evading these provisions, and it cannot be said that they are satisfactory or effective. Yet undoubtedly the general requirements of approval by the Highway Commission have had a very beneficial effect in securing better standards of construction and better supervision of the work.

KANSAS: Motor vehicle revenues returned to the counties and townships must be spent for the construction, improvement, reconstruction and maintenance of roads and bridges, but the Highway Commission has had no jurisdiction over the expenditure of these funds since 1929.

KENTUCKY: None of the motor vehicle revenue derived in Kentucky is returned to local units of government. However, something similar ocurs in the distribution among the counties of what are known as Rural Highway funds. Two million dollars a year are set aside for Rural Highway Projects. A Rural Highway Department is set up within the State Highway Department to arrange State Highway work in each county with the various Fiscal Courts; the priority of construction of these Rural Highways is generally left to the Courts, although the Rural Highway Department has power of veto if it so desires. While this procedure is not actually a return of funds to local units, it has almost the same effect, and the State Highway Department keeps final control of the expenditures.

LOUISIANA: Motor vehicle revenues are not distributed to local units for highway purposes, the State having assumed responsibility for local work.

MAINE: Wherever State funds are used for highways, it is the intent of the law that the work shall be under direction and control of the State Highway Department. As an example of the many different activities worked in cooperation with towns, the latter each year receive $670,000 for construction on roads accepted as third class highways by the State Highway Commission. Work must be done on locations approved by the Commission on petition of the municipal officers of the town. All projects are laid out and supervised by the State engineering force and financed by the town, which submits proper vouchers for reimbursement. The State supervisor certifies that the work has been performed in accordance with specifications.

MARYLAND: Methods of motor vehicle revenue allocation to the counties and Baltimore City are determined by a number of different laws and appropriation bills. Local units must apply these funds to highway purposes, and while State supervision and approval may be the measure of control in some instances, in others the State actually does the work.

MASSACHUSETTS: In general, all acts distributing motor vehicle funds to local units of government provide that the money shall be used for road improvement and authorize the Department of Public Works to pass upon the projects and supervise the work. However, no State control over local expenditures has been provided in the case of the 1940 and 1941 annual distributions of $9,600,000.

MICHIGAN: Reports must be made by the counties and municipalities to the State Highway Commissioner showing certain priorities required by law in the expenditure of State motor vehicle revenues by local units of government. Such priorities, for example, include debt retirements of various issues, and amounts passed on to cities. The law also requires that a strict accounting of the use of highway monies be made to the county board of supervisors and forwarded to the State Auditor General, defining amounts spent for construction, maintenance, right of way, and so on. The Auditor General is required to make periodic audits of the county books, thus assuring their accuracy. The reports to the Highway Department are of little value in assuring proper control of expenditures, while the reports to the Auditor General, showing what has actually been done with the money, come closer to being a desirable report. However, neither permit improvement in methods of operation.

MINNESOTA: One-third of the money received from the gas tax is allocated to the counties annually for use on county aid roads. The Highway Department has no control over the expenditure of this money. The law does provide, however, that all this money must be spent for highway construction or maintenance.

In Minnesota there is another source of State revenue distributed to the counties, this being known as the State-aid fund collected from a State-wide property tax of one mill per dollar of assessed valuation. At the present time this tax produces approximately $1,200,000 per year. The expenditure of this money by the counties is under supervision of the State Highway Department. The county must make the expenditure first and then report it to the Commissioner of Highways for approval. If the

work is approved by the Commissioner or his authorized representative, the county is reimbursed.

MISSISSIPPI: The State Highway Department has no jurisdiction over funds expended by the counties, nor the location of any county projects involving State funds. There is a State law which requires the counties to apply a certain part of gas tax receipts toward the retirement of outstanding highway obligations.

MISSOURI: No motor vehicle revenues are distributed to local units of government.

MONTANA: Revenue from the sale of license plates goes to the county in which the licenses were taken out. The State Highway Department has no control over the expenditure of these funds by the counties. No other local units of government receive motor vehicle revenue.

NEBRASKA: The Department of Roads and Irrigation has no power to require that money distributed to local governments be spent for roads and streets, nor can it pass upon local highway projects or supervise the work.

NEVADA: No part of motor vehicle tax revenue is distributed to local units of government.

NEW HAMPSHIRE: There appears to be ample authority vested in the Highway Commissioner under existing laws to designate projects and supervise work performed by local jurisdictions with motor vehicle revenue. There are instances where this authority has not been fully exercised, however, so that results are not quite as effective as might be desired.

NEW JERSEY: State motor vehicle tax money distributed to the counties and municipalities is under the supervision of the State Highway Commissioner and can be spent only after detailed approval by the Department. Such funds cannot be used for any other purpose than the improvement, maintenance and repair of highways and bridges.

NEW MEXICO: The State Highway Commission has no control over the expenditure of motor vehicle tax revenues allotted to the counties.

NEW YORK: Each county has a designated system of highways upon which its share of motor vehicle revenues must be spent. This system is subject to approval by the State Superintendent of Public Works. Before any State aid money can be spent by the county a project statement showing the place and the length and type of construction must be submitted and approved by the State Superintendent. The same procedure applies to county road maintenance, with the further provision that in the event of unsatisfactory maintenance, the Superintendent of Public Works may withhold approval of any proposed expenditures for construction and reconstruction in the county. The towns receive a limited amount of direct State aid, averaging about $3,000,000 per year, over which the State has practically no supervision.

NORTH CAROLINA: All highways are under State control, and no motor vehicle revenue is distributed to local units.

NORTH DAKOTA: The Highway Department has no legal authority over the expenditure of motor vehicle revenue distributed to local units of government.

OHIO: State motor vehicle funds are allocated to counties, municipalities and townships without any provision for State control or supervision in their expenditure. However, there is one exception, in case the local subdivision participates in the cost of an improvement on the State highway system, either in the county or municipality. The State then has complete control of the project and must prepare or approve the plans, let the contract, and supervise the work.

OKLAHOMA: The State Highway Department has no control over motor vehicle funds allocated to counties and municipalities except in cases where the counties participate (always voluntarily) with the State Highway Department in an improvement on any part of the State Highway System. In that event the State has complete control of the project, prepares the plans, lets the contract and supervises the work. There is one exception, however, where the State has permitted municipalities to prepare W.P.A. projects for work on the State highway system and has contributed to such work without retaining any further supervision than to check the plans and look over the work and materials furnished by the State Highway Department.

OREGON: The only local units receiving shares of State motor vehicle revenue are the counties. The laws governing distribution stipulate that funds are to be used solely for

road purposes, including the payment of interest and principal on road bonds, but it does not empower the State Highway Department or any other State agency to enforce that requirement or to exercise any control over the expenditure of the moneys or the planning and performing of the work.

PENNSYLVANIA: The 1939 Act of the Legislature allocating motor fund money for second-class township roads provides that expenditure shall be made in accordance with the rules and regulations of the Department of Highways. In addition, a part of gasoline tax revenues are refunded to the counties, and this can be used only for highway purposes. Many counties made grants of this money to townships, boroughs and cities under provisions of the County Aid Act, which requires that all work done under the Act shall be subject to the inspection and approval of the Department of Highways. This means that the work must conform to satisfactory standards and specifications.

RHODE ISLAND: The only assistance to local units in Rhode Island is a small allotment to the towns for maintenance of roads which are on the State Highway System but not constructed or maintained by the State. This amounts to only $200 per mile annually, but the money must be spent subject to State approval on specified roads in the system.

SOUTH CAROLINA: The law provides that motor vehicle funds distributed to the counties, comprising revenues from one cent of the gas tax levy, shall be used for road purposes, but leaves the expenditure entirely to the discretion of county officials.

SOUTH DAKOTA: By State law motor vehicle funds distributed to counties and townships must be used for highway purposes. The State Highway Commission, however, has no jurisdiction over the spending of this money, the matter being left to the discretion of county and township boards. It appears that these State funds are used for highways, but with no definite construction program in mind, and principally for minor improvements and maintenance. There is no provision that any part of this money shall be used to match Federal aid for secondary roads.

TENNESSEE: The 2-cent gas tax fund distributed to the counties may be administered by the State Highway Department only in those counties in which the County Courts pass resolutions requesting such action.

TEXAS: Statutes allocating motor vehicle revenues to the various counties provide that funds be spent only for the construction and maintenance of county roads. These expenditures are made under the control and direction of the several county commissioners' courts, and the State Highway Department does not in any manner supervise the expenditure of such revenues, nor does it have authority to pass upon county highway projects. When any county does not have an engineer, it is entitled to call upon the State Highway Department for assistance, but this rarely includes supervision of the work.

UTAH: The State Roads Commission's procedure in handling the distribution of motor vehicle revenues to counties and cities is to require a cooperative agreement with the local authority with respect to the location and character of the work to be done, and various other details. Until these cooperative agreements are executed, no funds are released for expenditure by these local units. Accounts are also required showing the purpose, character, location and cost of all work. The law permits the Road Commission to perform the work directly, if so requested by the local authorities, and in quite a few instances this is done.

VERMONT: Direct supervision over expenditures of State money on local highway projects is provided except in the case of $750,000 which goes to the towns for town roads. The law stipulates that such funds be spent under supervision of a committee of five, including three town selectmen, the town road commissioner, and the State's district highway Commissioner. The district Commissioner supervises practically all the work, and the towns are reimbursed after submitting time sheets and receipted bills covering the work performed. This gives the State sufficient authority to see that all money is spent for highways.

VIRGINIA: The State Highway Department has complete control over the expenditure of motor vehicle revenue except in the case of three counties which have not transferred their road operations to the State. In these three counties the money is expended under control of the County Board of Supervisors.

WASHINGTON: Funds credited monthly to the counties and cities are paid out only in reimbursements of expenditures made by these jurisdictions for proper road purposes, and in the case of cities, for construction and repairs only. Reimbursing vouchers, properly supported by detailed evidence of expenditures,

must be submitted through the Director of Highways. With the reasonable supervisory authority of the State, efforts of the political subdivisions are being coordinated and systematized to produce a comprehensive highway plan.

WEST VIRGINIA: All highways are under State control, and no part of motor vehicle revenue is returned to local units of government.

WISCONSIN: Supervision of the Highway Commission over the expenditure of motor vehicle revenue by local units of government is practically nil. The law in connection with aids to townships provides that expenditures be reported to the county boards and to the State Highway Commission. Very few townships make such reports, and the State has no way of compelling them to do so. The expenditure of aids to counties and cities is not required to be reported to the State Highway Commission. In general it is believed that such revenue allocated to local units is used for highway purposes.

WYOMING: The Wyoming State Highway Department has no jurisdiction over the expenditure of motor vehicle revenue distributed to local units of government. The law provides that such funds be spent for highway construction, and it is up to the State Examiner to find out if this provision is being followed. Although this has been difficult, the legislature has passed a county budget law which is expected to result in a closer check on the counties in their expenditure of motor vehicle money. At the present time the Department is doing missionary work in an effort to have the counties spend State aid funds on State-county cooperative projects, in which case the State has full control over the work. Progress is being made in this direction.

TREND IN THE PROVISION OF HIGHWAY SERVICE

Replacements and Maintenance

A satisfactory standard of highway service involves not only adequate physical plant, but also its proper maintenance and efficient operation. For satisfactory transportation can no longer be conceived merely as the economical movement of passengers and freight, but rather as safe movement with speed and comfort.

In the provision of highway transportation it is fairly evident that emphasis has shifted from expansion of plant to modernization, replacement and maintenance. For example, Federal aid projects today in nearly 60 per cent of all cases are located where previous Federal work has already been done, while much of the remaining work has been reconstruction of earlier undertakings by the States and counties. The increasing importance of the maintenance bill also is evident, State highway expenditure figures since 1921 revealing that while capital outlay in 1938 was less than in

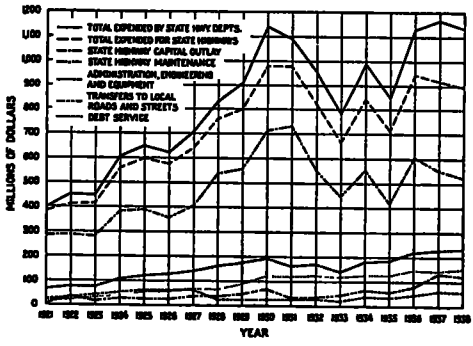


Figure 5. State Highway Expenditures 1921-1938

1928, maintenance expenditures had increased by close to $100,000,000 (see Fig. 5 and Table 6). Comparing the years 1932 and 1937, when capital outlays were identical at $551,000,000, maintenance expenditures had increased from $169,000,000 to $228,000,000.

Traffic Efficiency

The newest development in highway expenditure programs, however, has been a growing attention to efficient movement of traffic. Experience on the State highway system of California, for example, provides specific data on expenditures for services directly related to public convenience, safety and comfort. It is revealed that these services, including

TABLE 6

STATE HIGHWAY EXPENDITURES[1], 1921-1938

Year	Total expended[2] by state highway departments	Total expended[2] for state highways	State highway capital outlay	State highway maintenance	Administration engineering, equipment	Debt service	Transfers to local roads and streets
1921	$ 397,483,000	$384,381,000	$284,302,000	$ 64,833,000	$24,828,000	$ 17,067,000	$ 6,451,000
1922	451,775,000	410,901,000	287,461,000	75,340,000	32,046,000	22,257,000	34,669,000
1923	447,362,000	414,807,000	279,992,000	75,329,000	41,963,000	32,473,000	17,603,000
1924	605,665,000	559,601,000	382,335,000	104,806,000	50,958,000	38,895,000	28,669,000
1925	649,125,000	597,902,000	389,327,000	119,303,000	61,110,000	53,808,000	25,575,000
1926	621,744,000	576,016,000	356,174,000	125,617,000	60,534,000	55,569,000	23,848,000
1927	707,179,000	640,494,000	404,217,000	138,783,000	62,213,000	65,974,000	35,989,000
1928	830,264,000	759,318,000	538,043,000	159,807,000	24,516,000	64,653,000	35,885,000
1929	910,485,000	799,876,000	557,400,000	173,060,000	23,580,000	88,218,000	45,791,000
1930	1,139,676,000	979,997,000	713,117,000	191,683,000	24,528,000	120,172,000	66,897,000
1931	1,091,009,000	979,592,000	730,954,000	160,980,000	25,794,000	119,140,000	32,969,000
1932	955,446,000	816,765,000	551,445,000	169,479,000	26,547,000	123,671,000	34,324,000
1933	782,006,000	666,061,000	446,841,000	138,829,000	19,869,000	116,828,000	42,797,000
1934	991,774,000	843,631,000	554,278,000	178,999,000	37,469,000	125,881,000	64,815,000
1935	848,355,000	713,066,000	416,412,000	184,458,000	34,472,000	123,674,000	56,938,000
1936	1,131,151,000	947,085,000	607,284,000	219,202,000	45,353,000	145,633,000	77,541,000
1937	1,166,706,000	918,786,000	551,979,000	227,877,000	60,354,000	141,488,000	130,477,000
1938	1,135,122,000	895,132,000	523,738,000	232,388,000	55,984,000	150,033,000	121,262,000

[1] Data compiled by the Public Roads Administration, Tables F-2 (1921-33) and SF-2 (1934-38).

[2] These columns include sums which may not appear in the various items listed, such as expenditures for State police and non-highway purposes.

pavement markings, signs, traffic control devices, roadside development, snow removal and ice control have trebled in cost since 1930. In the next two years the California Division of Highways expects to use approximately 12 per cent of all maintenance funds to comply with the growing demands for a better quality of highway service.

Other States in their biennial reports are emphasizing this trend toward improving highway transportation by more attention to operations on the existing plant. To mention a few specific cases, Vermont reports that winter maintenance has become an increasingly costly item and that more money is spent to sand the roads than to plow them.[35] According to the Highway Department, however, "the more satisfactory travel conditions resulting justify the expense and meet the motorists' demand." [36] The Vermont report further emphasizes the public service factor by noting that the marking of all bituminous roads with center lines was believed to have "met with the hearty approval of the traveling public." From Minnesota comes evidence of the same trend in a statement of the Commissioner of Highways: "Public Service, the primary function of which is to control traffic and promote safety, which includes such items as sign erection, center line marking, highway patrol and other safety functions, required an expenditure (1937-38) of $1,478,000." [37]

Roadside improvement activities during the past several years provide additional testimony of increasing expenditure for safer and more attractive highways. By October 31, 1939, the minimum one per cent Federal aid provision for roadside development in the States had resulted in projects costing approximately $16,000,000, of which $10,000,000 were Federal funds. Emphasis on roadside programs has been directed not only to purposes of appearance, but to erosion control, snow control, roadside parks, turnouts, and tourist accommodations. As one example of progress in this type of highway service, Michigan reports that the number of roadside parks has increased since 1933 from 12 to 131, and the number of roadside picnic tables from 100 to 2,550.[38] A system has also been developed for trimming trees to secure proper underclearance and to provide adequate visibility at curves and intersections.

Safety considerations are properly the principal factor in the effort to furnish a better quality of highway transportation, being an important consideration in winter maintenance and roadside development as well as in traffic engineering, signs, signals, markings, highway patrols, accident reporting, vehicle inspections and driver regulation. One of the most widespread developments for highway safety has been that of the State highway patrols, personnel having increased from 5,407 in 1936 to nearly 10,000 in 1939.[39] Allotments of State highway user tax receipts for the purpose of supporting the patrols have increased from $8,751,000 in 1934 to $23,406,000 in 1938. In the 1939 legislative sessions these trends were continued. Tennessee increased from 5 to 10 per cent the share of motor vehicle funds available for the highway patrol, while in Connecticut 75 per cent of the cost of operating the State patrol

[35] In 1936-37, for the U. S. as a whole, States and counties alone spent $17,000,000 for snow removal and ice treatment on 217,000 miles of highways.

[36] Vermont Highways, Eighth Biennial Report, State Highway Board, 1936, p. 25.

[37] Commissioner of Highways, Biennial Report, 1939, p. 51.

[38] Seventh Biennial Report, State Highway Commissioner, 1939.

[39] Highway police agencies of 300 or more officers are operated by 10 States: Illinois, Indiana, New Jersey, New York, Pennsylvania, Iowa, Massachusetts, Michigan, California, and Texas. Pennsylvania has the largest patrol, with 1,596 officers.

will be derived from the highway fund. Minnesota increased the membership of its patrol from 100 to 108 for 1939, and to 116 thereafter.

A forward-looking view of safety responsibilities in the highway transport field was presented by Mr. E. W. James at the meeting of the Highway Research Board two years ago.[40] This report enumerated some of the auxiliary services which may have to be performed to secure the greatest results in the direction of safer and more efficient highway operation, citing as parallel examples what has already been done in the case of rail, maritime and air transportation. The cost of providing facilities for the functions conceived, such as driver training and licensing, vehicle inspection, accident records and highway patrol, was estimated at between $76,000,000 and $124,000,000 for capital investment and between $169,000,000 and $366,000,000 annually for operation, depending upon the extent of development. "These services," it was stated, "will become a third grand division of highway costs, added to the division of construction and maintenance."

Urban Highway Service

Financial and administrative developments tending toward the improvement of highway transportation facilities in cities have already been mentioned. Among these have been State assistance in the form of motor vehicle revenue allotments and work done on city trunkline extensions; and Federal assistance through direct emergency appropriations to municipalities and by the removal of restrictions to the urban use of regular Federal aid appropriations. At the same time there is the current possibility of further assistance to the cities in plans for the acquisition of land and the construction of express routes.

These urban requirements and recent plans to fulfill them have received widespread emphasis. There has been an equally universal neglect, however, of one essential factor in urban highway service; namely, the provision of terminal facilities. This neglect is particularly harmful to a short-run agency of transportation which cannot attain its natural advantage when desirable origins and destinations are denied. At the same time the effects of other measures to improve city travel can hardly be successful while a primary cause of congestion continues to be overlooked. For the storage of automobiles on expensive city pavements constructed for vehicle movement is certainly a factor in the lack of street space for urban traffic, as is also the cruising and maneuvering of hopeful parkers.

Today the parking lot has become a municipal utility.[41] In 1937 at least 33 cities in 17 States had begun to take the problem seriously. In some cases public property has been used for parking lots, while in others real estate has been taken over for nonpayment of taxes, or loaned to the city by the county or State. Land has been acquired by excess condemnation and by lease from private owners. One city has acquired parking space by condemning convenient rearage instead of expensive frontage. Some of these lots are established on a self-liquidating basis by the charging of fees, but most are provided "free" by assessing property in the benefited areas.

Further promotion of parking accommodations has been provided by law, a number of cities having passed legislation requiring newly constructed theatres, apartments, stores and places of amusement to provide on their premises a specific square footage of parking area for each person to be accommodated. In the

[40] E. W. James: Accident Records and Traffic Regulation, *Proceedings,* Highway Research Board, Vol. 17, p. 435.

[41] See "The Parking Problem in Central Business Districts," Public Administration Service, Publication 64, 1938.

1937 session of the State legislature, Kansas authorized cities of the first class to establish benefit districts for acquiring and improving parking stations for public use, permitting special improvement bonds to pay the cost, while California likewise provided for the establishment of parking places in municipalities. In Michigan parking facilities were added to the activities which may be financed under the revenue bond act.

Although considerable activity on a small scale is being directed toward the provision of parking space, recognition of the problem has been so late that now only drastic methods can provide a remedy. Whatever these may be, it appears reasonable that policies intended to permit easier entrance into cities should be supplemented by provision of parking accommodations after arrival; and that measures for acquiring adequate rights of way should include the acquisition of land for that purpose.

Highway Service Trends—Conclusion

The conception of highway transportation has developed in the past few years to include not only an adequate physical plant, but also efficient operation to provide motoring convenience, comfort, and safety. Appraisal of what has already been done and what remains to be done, especially in regard to safety operations and parking facilities, is convincing indication of further uptrends in expenditure for a higher quality of transportation service.

GENERAL CONCLUSION

In the financing of highways there are logical criteria by which revenue sources, tax rates and expenditure objectives may be intelligently determined. The problem is, in fact, an equation of supply and demand requiring solution in such a manner that financial needs and user revenues may coincide to permit an optimum in highway service and highway utilization. The amount of money available in a given year should depend upon established requirements based on the planned attainment and perpetuation of standards considered desirable; and this amount, translated into charges payable by those making use of the facilities, would then determine a rate of tax. Consideration must further be given to probable effects of highway charges upon highway use and highway income, so that revisions and modifications may be made accordingly.

A planned highway program which has taken into consideration all the above factors should then indicate how much money is to be spent, where and for what purposes expenditure should be made, and from whom the necessary income may be derived. Today, however, the reverse is generally true; that is, predetermined financial policies dictate what shall be the highway program.

Obviously the first step in rearranging this relationship is the determination of a physical and financial plan for the highway system. For only the tangible expression of differences between present highway service and future highway objectives can furnish the necessary guidance for public opinion and legislative action. Without such factual guidance the highway financial program can never be more than the product of uninformed or inexpert opinion.

The ultimate goal of highway planning, as conceived in the current studies being undertaken by the State Highway Departments in cooperation with the Public Roads Administration, is the formulation of plans which may serve as a guide not only to the proper administration of highways, but to the enactment of legislation which will permit such administration on a continuing basis. Already several States have completed or are in process of completing these plans. That the preparation of similar material by other States is essen-

tial to sound financial practices should be amply demonstrated by the quantity of legislation and the quality which results without such guidance. The fact that 10,000 bills on highway transportation were introduced in 1939 legislative sessions, and that 1,200 of these were enacted into law, removes any doubt that further assistance from the Highway Departments in the formulation of highway laws is, from an economic viewpoint, compulsory.

DISCUSSION ON TRENDS IN HIGHWAY FINANCIAL PRACTICE

MR. BURTON W. MARSH, *American Automobile Association:* I would like to ask Professor Morrison if he can explain the trend in Michigan away from property taxes.

PROF. R. L. MORRISON, *University of Michigan:* I think the main reason is that the Michigan farmer is very firmly entrenched in the Legislature. For a number of years attempts were made to put the townships under county administration without success. Finally instead of a bill for administrative change a tax relief bill was proposed which went over with a bang.

MR. WILFRED OWEN, *Highway Research Board:* May I add that in Michigan a large amount of motor vehicle revenue is going to the counties. I think that half of all highway user revenue is being distributed to local units, so that naturally property can be relieved of that responsibility.

PROFESSOR MORRISON: All of the license fees go back to the counties where they are collected—seven-eighths to counties where collected and one-eighth divided among eight counties, in other words, Wayne County where Detroit is located helps out in the Upper Peninsula. The counties also get $6,000,000 of the gas tax.

MR. MARSH: I think greater attention must be given to the adding of terminal facilities in metropolitan areas. Surely if we are going to make it easier for persons to get into cities a good many of them are going to want to stop. The record shows that $250,000,000 was spent on municipal projects from 1933 to October 31, 1937. Is that rate increasing as the years go on?

MR. OWEN: No, a large part of it was emergency funds spent by P.W.A. and W.P.A. It is not part of the regular Federal Aid program.

MR. MARSH: The question of proper control of expenditures is exceedingly important. Mr. Owen pointed out that more States do not have control over local expenditure than have.

MR. L. S. TUTTLE, *Public Roads Administration:* Does the committee see any visible trend with respect to control: Is it going to be better or worse?

MR. OWEN: In the 1939 Legislatures there were more bills of an adequate nature passed on State control than in any other sessions. Also many of the States are now requiring budget reports from their counties and some States which have had legal provisions in the past are only now putting these legal controls into effect. The letters from the highway departments which provided this information are very interesting in that the State highway engineers in many cases have mentioned the fact that matters are improving in their States.

DEPARTMENT OF HIGHWAY TRANSPORTATION ECONOMICS

R. L. MORRISON, *Chairman*

REPORT OF COMMITTEE ON TRACTIVE RESISTANCE AND ALLIED PROBLEMS

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BIBLIOGRAPHY OF TRACTIVE RESISTANCE AND ALLIED PROBLEMS

SYNOPSIS

In this bibliography there are about 240 items arranged in chronological order. These data relate to four subjects, the road, the vehicles, the tires, and brakes, which are indicated by the letters A, B, C, and D, respectively. Various characteristics of these four subjects are discussed, such as skidding, surfaces, hill climbing, tractive resistance, air resistance, and so forth, which are all considered in outline form on one of the first pages. On the succeeding pages may be found the papers indicated by numbers which have been written on each characteristic of each subject. Then on each reference which is numbered, as before, according to chronological order, may be found letters a, b, c, d, and so forth, which indicate which subjects and which characteristics they cover.

CLASSIFICATION OUTLINE

DESIGNATION OF CONTENT

Subject		Characteristics
A	Road	a General
		b Surfaces
		c Coefficient of friction
		d Skidding
		e Cornering
		f Deceleration—stopping distances
		h Hill climbing
		j Tractive resistance
		k Rolling resistance
		l Grade resistance
B	Vehicle	a General performance
		d Skidding
		f Deceleration—stopping distances
		g Acceleration
		h Hill climbing
		i Fuel consumption—mileage
		j Tractive resistance
		k Rolling resistance
		l Grade resistance
		m Air resistance
C	Tires	a General performance
		b Tread surface
		c Coefficient of friction
		d Skidding
		e Cornering
		f Deceleration—stopping distances
		k Rolling resistance

D Brakes ....a General performance
d Skidding
f Deceleration—stopping distances

CLASSIFIED INDEX

ROAD

Aa Road—General
4–16–18–24–30–35–36–39–43–47–48–50–51–62–72–83–98–99–100–101–103–122–127–167–174–175–188–207–216–217–220–244

Ab Road—Surfaces
8–9–12–18–22–26–30–47–62–69–72–76–83–98–99–100–101–103–121–122–124–127–146–160–166–168–174–175–188–197–204–217–218–220–231

Ac Road—Coefficient of Friction
8–9–12–19–69–72–76–83–98–99–100–101–103–121–122–127–146–166–174–188–217–218–220–231

Ad Road—Skidding
8–9–12–30–69–72–76–99–100–101–146–160–166–168–174–196–197–204–218–231

Ae Road—Cornering
8–168–197

Af Road—Deceleration, stopping distance
30–51–69–168–196–197

Ah Road—Hill climbing
48–124–235

Aj Road—Tractive Resistance
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Ak Road—Rolling Resistance
19–24–26–62–98–244

Al Road—Grade Resistance
22–48–62–244

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Ba Vehicle—General Performance
2–3–4–6–7–10–14–15–16–18–20–21–23–24–25–27–30–32–33–35–36–39–40–41–43–44–45–47–48–50–51–53–54–62–72–73–80–81–83–87–92–98–99–100–101–103–106–110–111–113–115–116–117–118–120–122–125–126–127–130–131–132–134–135–142–144–148–149–150–155–157–167–174–175–179–188–189–191–198–207–216–227–228–240

Bd Vehicle—Skidding
8–9–29–30–69–72–73–74–76–81–99–100–101–104–105–119–120–130–133–134–146–151–160–166–168–174–196–197–203–219–222–231–233

Bf Vehicle—Deceleration, stopping distance
15–29–30–51–69–73–74–87–111–115–116–118–131–168–195–196–197–198–203–208–210–221–230

Bg Vehicle—Acceleration
15–16–40–45–97–111–113–115–116–118–120

Bh Vehicle—Hill climbing
7–48–110–111–113–115–116–117–118–124–132–150–235

Bi Vehicle—Fuel consumption, mileage
18–24–25–37–48–56–62–71–78–88–93–142–143–144–153–154–161–175–194–207–211–216–223–240

Bj Vehicle—Tractive resistance
3–10–14–16–24–26–36–37–40–50–51–56–66–93–98–113–117–121–122–123–127–150–175–236–244

Bk Vehicle—Rolling resistance
24–26–62–88–92–98–123–244

Bl Vehicle—Grade resistance
48–62–123–244

Bm Vehicle—Air resistance
1–11–17–24–28–31–41–42–46–49–57–59–60–61–63–68–71–73–75–77–78–79–80–85–86–98–102–108–112–123–126–128–140–141–142–143–144–145–147–152–153–154–156–157–159–161–162–163–164–169–170–171–172–173–176–177–179–180–181–182–185–186–187–189–191–193–194–198–199–200–201–202–209–211–212–213–214–215–223–227–238–239–240–241–243–244

TIRES

Ca Tires—General performance
4–5–18–51–58–70–72–92–93–98–99–101–103–107–130–158–165–178–183–184–188–190–192–205–206–220–226–229–237

Cb Tires—Tread surface
13–18–69–70–72–82–98–99–101–103–107–139–146–188–190–204–220–226–232–234

Cc Tires—Coefficient of friction
13–19–69–72–82–94–98–99–101–103–105–109–114–139–146–178–188–190–220–232

Cd Tires—Skidding
13–69–72–82–94–99–101–105–130–146–151–204–232

Ce Tires—Cornering

Cf Tires—Deceleration, stopping distance
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Ck Tires—Rolling resistance
19–38–52–58–65–67–88–92–98–109–139–158

BRAKES

Da Brakes—General performance
51–72–73–87–90–91–103–129–130–176–178–198–224–225

Dd Brakes—Skidding
69–72–73–74–90–91–129–130–146–151–203

Df Brakes—Deceleration, stopping distance
51–69–73–74–87–90–91–195–198–203–208–210–221

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54. Ba "Chassis Friction Losses." E. H. Lockwood, p. 384, S.A.E. *Transactions,* 1922, Part II.

56. ABij "Motor Truck Tractive Resistances on Road Surfaces." T. R. Agg. *Engineering News-Record,* V. 89, No. 23, Dec. 7, 1922, pp. 982-985, 9 figs. Elimination tests of measuring devices prove superiority of space-time recorders; gasoline consumption measured in relation to tractive resistance.

57. Bm "Auto Air Resistance Measured in Wind Tunnel." *Auto. Industries,* V. 47, No. 23, Dec. 7, 1922, p. 1137. Tests made at Zeppelin Works show that air resistance of modern automobiles is materially less than would be concluded from coefficients found in engineering text books; advantages to be derived from complete streamlining of cars.

58. Cak "Power Losses in Pneumatic Tires." O. Enoch. *Motorwagon,* V. 25, No. 28, Oct. 10, 1922, pp. 529-531, 3 figs. Reply to S.A.E. Jour. paper giving measurement results of rolling resistance of cord and fabric tires which Prof. Lockwood, the author, claims are in disagreement with tests made in writer's laboratory and other German tests.

59. Bm "Calculation of Energy Output of Automobiles with Special Regard to Air Resistance." P. Jaray, *Motorwagon,* V. 25, No. 29, Oct. 20, 1922, pp. 551-559. Results of measurements of air resistance of automobiles with different types of bodies, made in large wind tunnel of Zeppelin Works at Friedrichshafen. Air resistance was found to be considerably less that has been generally assumed heretofore. Advantages of streamline form.

60. Bm "Streamline Bodies." Ernst Neumann-Nearder. *Motorwagon,* V. 25, No. 22, Aug. 10, 1922, pp. 418-420. Advantages and disadvantages of changing present shape of bodies.

61. Bm "Air Resistance Investigations on Automobile Models." W. Klemperer. *Zeit. für Flugtechnik ü Motorluftschiffahrt,* V. 13, No. 14, July 31, 1922, pp. 201-206. Based on ideas and designs developed by P. Jaray, series of systematic tests were carried out in large wind tunnel of Zeppelin works to determine what advantages could be gained by building automobile bodies along aerodynamically favorable lines; investigations of Jaray streamline cars and other models.

62. ABabikl "Surfaced Roads Would Save $23,000,000." A. W. Campbell. *Contract Record,* V. 36, No. 25, June 21, 1922, pp. 581-582. This amount would be saved to motorists in decreased gasoline consumption alone, aside from lower repair costs; effect of road surfaces and grades on tractive resistance.

63. Bm "Streamline Autos." P. Jaray. *Motorwagon,* V. 25, No. 17, June 20, 1922, pp. 333-3336, 7 figs. New type of construction to reduce air resistance; results of experiments made in Zeppelin wind tunnel; fuel consumption, etc.

65. Ck "Power Losses in Auto Tires." Holt and Wormeley. Bureau of Standards, May 20, 1922, Paper No. 213. Technologic Paper.

66. Bj "Resistance to the Translation of Motor Vehicles." T. R. Agg. Iowa State College of Agriculture and Mechanic Arts, Official Pub., Bul. No. 64. V. 20, No. 53, May 31, 1922, 32 pp., 18 figs. Deals with abortive attempts to measure so-called tractive resistance and describes methods finally adopted. Results seem to show that rolling resistance is about half of total resistance to translation when vehicle with good tires is operated on hard, smooth road surfaces, and resistance due to impact is very small on such surfaces.

67. Ck "Power Losses in Automobile Tires." W. L. Holt and P. L. Wormeley. U. S. Bureau of Standards, Technological Papers, V. 16, No. 213, May 20, 1922, pp. 451-461, 8 figs. Relates to power loss or energy dissipated as heat in automobile tires when operating under different conditions of axle load, inflation pressure, speed, and tractive effort.

68. Bm "Rear Wheel Dynamometer Tests and Their Significance to the Engineer." Herbert Chase. *Auto. Industries,* V. 46, No. 16, Apr. 20, 1922, pp. 859-868. Description and data from apparatus in Mason Laboratory, Sheffield Scientific School, Yale University. Includes comments on rolling friction, wind-resistance, tractive effort, and fuel economy.

69. ABCDbcdf "Bureau of Standards Makes Definite Checks on Brake Performance." Herbert Chase. *Auto. Industries,* V. 49, No. 22, Nov. 29, 1923, pp. 1092-96. Relative effectiveness of various arrangements and operating means are graphically recorded; use of front-wheel type does not double rate of deceleration, surprising values for friction coefficient between tire and road obtained.

70. Cab "Dynamometer Tests of Auto Tires." W. L. Holt and P. L. Wormeley. U. S. Bureau of Standards Technologic Paper No. 240, Sept. 24, 1923, 559-579. Details of power-loss tests on a large number of tires, and comparative results for the 3½, 4, and 5" sizes, both cord and fabric; analysis of effect of different parts of tire on power loss; points out and gives examples showing how dynamometer tests may be used as a help in studying details of tire design.

71. Bim "Comparative Trial Trips with Jaray Streamline Automobile." R. Conrad, *Motorwagon,* V. 26, No. 23-24, Aug. 31, 1923, pp. 355-63. Results show economic advantages of Jaray car over standard type automobiles.

72. ABCDabcd "Quality of Brakes has Marked Effect on Capacity of Highways." W. S. James. *Auto. Industries,* V. 49, No. 9, Aug. 30, 1923, pp. 412-14. Minimum stopping distance is usually controlled by friction between tires and road and proportion of weight on wheels which are braked. Slippery road surfaces and unsafe brakes reduce number of vehicles per hour roads can handle.

73. BDacdfm "Distances Required for Stopping." P. M. Heldt. *Auto. Industries,* V. 48, No. 6, Feb. 8, 1923, pp. 280-81. Application of brakes produces effect of transferring weight from rear to front wheels; stopping time less with 4-wheel type; effect of air resistance.

74. BDcdf "Skidding of Vehicles in Quick Braking." Ottorino Sessini. *Ingegneiria,* V. 2, No. 4. Apr. 1, 1923, pp. 87-88. Discusses theory of skidding and calculates forces in question.

75. Bm "Streamlining a Closed Car." *Autocar,* V. 50, No. 1428, Mar. 2, 1923, pp. 354-6. Details of streamline. Farmans, both open and closed, and other examples.

76. ABbcd "Theory and Advantages of Balanced Brake Forces." Geo. L. Smith. S.A.E. *Journal,* V. 14, No. 2, Feb., 1924, pp. 111-116. Describes two methods of brake application in use in U. S., and expounds theory of balanced brake forces; practical applications of equalizing mechanism used in road tests of automobile; tests on wet pavements with study of skidding effects and skid-checking means; results of tests on hills and effects of speed and pressure.

77. Bm "The Automobile in Air Currents." E. Rumpler. *Zeit für Flugtechnik ü Motorluftschiffahrt,* V. 15, No. 3-4, Feb. 26, 1924, pp. 22-25. Discusses progress made in recent years in application of airplane theories to automobile design; introduction of streamline automobile developed by author in 1921; results of experiments carried out at Aerodynamic Experimental Station in Göttingen.

78. Bim "A Streamlining Development." *Autocar,* V. 52, No. 1486, Apr. 11, 1924, pp. 662-4. Unconventional body designed by a Danish engineer, P. Forostovsky, Jr., and fitted to a 2-liter Bignan chassis; head resistance diminished, resulting in increased speed and reduced gasoline consumption; but with no loss of comfort.

79. Bm "The Rumpler Cars, 1924." A. Conrad. *Motorwagon,* V. 27, No. 34, Dec. 10, 1924, p. 673-684. Describes latest model of Rumpler streamline car, first model of which was developed in 1922.

80. Bam "The Future High-Speed Automobile." A. Persu. *Allgemeine Automobile Zeitung,* V. 25, No. 18, May 3, 1924, pp. 25-26. Author states future high-speed car will be streamline type, in which all 4 wheels are built within streamline form, the two driving wheels will be less distance apart than front wheels, passenger seats will be in front and power plant in rear.

81. Bad "Testing Wheel Slip." *Autocar,* V. 52, No. 1474, Jan. 18, 1924, pp. 96-98. Results of trials with special instrument evolved to register action of differential.

82. Cbcd "Two Notable Tyre Developments." *Motor Transport,* (London) V. 39, No. 1017, Aug. 25, 1924, pp. 243-4. Low pressure pneumatic and new design of solid tire that show phenomenal non-skid capacity.

83. ABabc "Tractive Resistance and Related Characteristics of Roadway Surfaces." Bul. No. 67, 1924, Iowa State College, Eng. Exp. Station. An investigation conducted by the Iowa Engineering Experiment Station in co-operation with the U. S. Bureau of Public Roads and Iowa Highway Commission.

85. Bm "Wind Resistance of Motor Vehicles." L. E. Conrad. *Public Roads,* V. 6, No. 9, Nov. 1925, pp. 203-6. Progress reports of tests conducted by Kansas State Ag. Coll. in co-operation with Bur. of Pub. Roads. Resistance of full-sized cars measured in wind tunnel; effect of size of tunnel and other test conditions; complete tests on 15 cars; Buick touring car was found to have highest wind resistance; Ford roadster the lowest; develops equation for calculation.

86. Bm "Full Scale Wind Tunnel." E. R. Dawley. *Aviation,* V. 19, No. 8, Aug. 24, 1925, p. 212. Wind tunnel of Kan. State Ag. College, designed primarily for determining aerodynamic resistance of automobile, but has several features in it of interest to aeronautical engineers.

87. BDaf "Factors Determining the Minimum Stopping Distance of an Automobile." H. H. Allen. S.A.E. *Journal,* V. 17, No. 2, Aug. 1925, pp. 192-4. Summarizes and explains derivation of some of the relations that are essential to a correct determination of possible deceleration; all variables that are of significance or of sufficient magnitude to affect appreciably performance of a car under a given set of conditions of vehicle or of environment have been included.

88. BCik "Effect of Tire Resistance on Fuel Consumption." W. L. Holt and P. L. Wormeley. Bureau of Standards Technologic Papers, V. 19, No. 283. Apr. 6, 1925, pp. 213-23; also (abstract) in *Auto. Industries,* V. 52, No. 21, May 21, 1925, p. 902-4. Result of investigation shows that rolling resistance of balloon tires is slightly higher than that of high-pressure cord tires, but substantially lower than that of high-pressure fabrics; however, ability of tire to absorb road shocks and its durability are more important than slight effect it may have on fuel consumption.

90. Dadf "Braking of Automobiles." E. Essers. *Automobil-Rundschau,* V. 28, No. 16 and 17, Nov. 3 and Dec. 1, 1926, pp. 373-6 and 403-6. Calculation of shortest braking distance by means of nomogram; effect of rolling and sliding friction.

91. Dadf "Calculation of Car Travel with Brakes Applied." A. Stadie. *Motorwagon,* V. 30, No. 4, Feb. 10, 1927, pp. 64-73. Forces which have decelerating or braking effect on vehicle are external resistance, internal natural resistance, and resistance caused by braking; first two forces, even though small in comparison with resistance through braking. must be taken into consideration in accurate calculation.

92. BCak "Measuring Running Resistance of Automobiles." E. Essers. *Automobil-Rundschau,* V. 29, Nos. 18, and 19, Sept. 15 and Oct. 1, 1927, pp. 359-64 and 384-90. Report of experimental study, done at automobile lab. of Aachen (Aix-la-Chapelle) Institute of Technology; method consisted in keeping engine idle and causing auto to coast on normal road; rolling resistance with various types of cord and balloon tires.

93. ABCaij "The Motor Truck Tire in Its Relation to the Vehicle and to the Road." J. A. Buchanan, S.A.E. *Journal,* V. 20, No. 4, Apr. 1927, pp. 469-77. Presents some important data that have resulted from researches by the Bur. of Public Roads; general view is given of effects of vehicle type, wheel load, tire equipment. and road conditions on gasoline consumption, tractive resistance and impact relations.

94. Ccd "Resistance of Tires to Skidding." R. H. Baldock. *Rubber Age,* 44: 28, Oct., 1928.

97. Bg "Acceleration." E. C. Wadlow. *Auto. Engineer* V. 18, No. 249, Dec. 1928, pp. 440-2, 482-6. Comparison of conditions on road and on test bed; resisting torque and tractive resistance on road; torque-speed characteristics of friction, air, hydraulic, and electric dynamometers; from viewpoint of both resistance law and inertia of system, electrical swinging field dynamometer stands out as being by far best adapted to making acceleration tests previously outlined.

98. ABCabcjkm "Tractive Resistance of Automobiles and Coefficients of Friction of Pneumatic Tires." T. R. Agg. Iowa State College, Official Pub., Eng. Exp. Stat., Bull. No. 88, May 2, 1928, 60 pp., 50 figs. Report deals principally with investigations of certain actions and reactions between road surfaces and tires used on self-propelled vehicles; measurements of air resistance on automobiles by method of mounting auto. on dynamometer carried by flat car and to push car ahead of electric locomotive; coefficient of friction between auto. tires and road surfaces.

99. ABCabcd "Causes of Skidding" Institute of Municipal and County Engrs. *Journal,* V. 55, No. 26, June 26, 1928, p. 12. Investigations by British Ministry of Transport in causes of skidding and means of prevention; building of arterial roads in greater London has furnished opportunity of employing large variety of surfacing materials; skid tests; small portable machine designed and constructed for measuring coefficients of friction between rubber tires and road surfaces commonly used in Greater London areas; consideration of design of vehicle; special model vehicle has been constructed to carry out experiments to determine effect of stability.

100. ABabcd "Investigations into Causes of Skidding and Means of Prevention." *Roads and Road Construction,* V. 6, No. 64, Apr. 2, 1928, pp. 122-123. List is given showing wide range and distribution of materials; skid tests; vehicles; consideration of design of vehicle.

101. ABCabcd "The Causes of Skidding and Means of Prevention." *Surveyor,* V. 73, No. 1885, Mar. 9, 1928, pp. 303-304. Research necessary to determine respective parts played by materials and methods of road construction, and design of vehicles, tires, brakes, etc.; list shows wide range and distribution of materials; skid tests; small portable machine designed and constructed at Nat. Research Lab. for measuring coefficients of friction between rubber tires and various road surfaces.

102. Bm "Kansas Tests Give New Value for Air Resistance Coefficient." *Auto. Industries,* V. 58, No. 18, May 5, 1928, pp. 692-3. Tests of Eng. Exp. Stat. of Kan. State Ag. College; experiments conducted in specially constructed wind tunnel, show coefficient for passenger cars to be .0024 and for trucks .0025; racers also tested; wind tunnel features, differences in tunnel and on road kept in mind; distribution of air flow.

103. ABCDabc "Solid-Tired Buses Easier to Brake, According to French Tests." *Auto. Industries,* V. 58, No. 14, Apr. 7, 1928, pp. 560-1. Able to stop in shorter distance than when equipped with pneumatic tires; brakes also prove to be more effective on standstone pavement than on asphalt or on wood blocks; comparison between different road pavements, different conditions of road surface, different types of tire and different states of wear of tires; tests made with four conventional-type of four-wheel, 28 passenger buses. Report published in "Le Poids Lourd."

104. Bd. "The Skidding of a 4-wheeled Vehicle on a Slippery Road." L. H. G. Dines. *Engineer,* V. 145, No. 3759, Jan. 27, 1928, pp. 94-95. At exhibition at Nat. Phys. Lab. at Teddington there was shown simple and effective model which demonstrated remarkable fact that if 4-wheel truck, unprovided with steering lock, be traveling on smooth surface and have its front wheels suddenly locked by means of a brake, there is no tendency for truck, as a whole, to spin round; it continues to travel in straight line as before, till sliding friction brings it to a stop.

105. BCcd "Stopping Distance of Vehicle Independent of Gross Weight." E. Favary. *Power Wagon*, V. 41, No. 283. July, 1928, pp. 5-6. Minimum stopping distance of motor vehicle when all wheels are provided with brakes; retarding force and turning effort; slippage between tires and road; effect of drum diameter in chassis dynamometer tests.

106. Ba "Commercial Vehicles on Test." *Modern Transport*, V. 20, No. 495, Sept. 8, 1928: 4-speed Chevrolet truck; hill tests, etc. V. 20, No. 496, Sept. 15, 1928: 30 cwt. Bear engine; hill climb, accel. and brakes, etc.

107. Cab "Pneumatic Tyres." W. H. Paull. *Motor Transport*, V. 47, No. 1240, Dec. 17, 1928, pp. 739-741. Magnitude of power losses in tires; area of contact; estimated wear. Abstract of paper presented before joint meeting of Instr. Rubber Industry and Inst. Auto Engrs.

108. Bm "Air Resistance of Automobiles." E. H. Lockwood. *Proceedings*, Highway Research Board, Vol. 9, 1929.

109. Cck "North Carolina Tire Tester." Shaw and Fontaine. *Proceedings*, Highway Research Board, Vol. 9, 1929.

110. Bah "Road Passenger Vehicles on Test." *Modern Transport*, V. 22, No. 560 and 562, Dec. 7, 1929, pp. 19-20 and Dec. 21, pp. 18-19. Road performance characteristics—of buses—hill-climbing tests, among other things.

111. Bafgh "Road Passenger Vehicles on Test." *Modern Transport*, V. 22, No. 556, Nov. 9, 1929, p. 31-2. Brief description of test of 6-cylinder Mandslay; good hillclimbing; suspension tests; brake and acceleration tests; technical details.

112. Bm "Science in Body Building." H. E. Thomas. *Autocar*, V. 63, No. 1775, Nov. 8, 1929, p. 1029. Description of coupé body, designed for 30 to 98 h.p. Vauxhall automobiles which weigh less than driver; insulation; ventilation; etc. And: to investigate behavior of air currents around body, scale model of car was made and extensive wind tunnel tests carried out under varying conditions.

113. Baghj "Performance Evaluation." *Auto. Engineer*, V. 19, No. 259, Oct. 29, pp. 387-8. Description of Tapley Performance Meter for determination of general performance of motor vehicles; object is to furnish direct reading evaluations of performance, giving measure in lbs. per ton that engine is exerting on driving wheels; methods of measuring top gear acceleration; hill-climbing capacity; tractive resistance, actual power output, and engine friction.

114. Cc "Friction Coefficient Between Rubber and Various Materials." R. Ariano. *Politecnico*, V. 77, No. 5, May, 1929. pp. 259-274. See also *translated abstract* in *India Rubber Journal*. V. 78, No. 10, Sept. 17, 1929, pp. 351-353. Methods and results of series of tests, made at engineering lab. of Societa Italiana Pirellis, to determine wear of rubber-tired wheels travelling over concrete, wood-block and other types of roads; summary of results of American and German studies.

115. Bafgh "Commercial Vehicles on Road Test." *Modern Transport*, V. 22, No. 554, Oct. 26, 1929, pp. 10-11. Description of performance 80 h.p. Mercedes-Berg crude oil chassis with load of 10 tons; ease of handling; hill test; brake test; perfect engine performance; power unit; fuel and oil supply; transmission arrangement.

116. Bafgh "Commercial Vehicles on Road Test." *Modern Transport*, V. 22, No. 549, Sept. 21, 1929, pp. 19-20. Description of test on new 1½ ton Morris commercial truck; ease of control, remarkable climbing performance acceleration tests.

117. Bahj "Commercial Road Vehicles on Test." *Modern Transport*, V. 21, No. 544, Aug. 17, 1929, pp. 21-22. Description of road tests of 26.9 h.p. Renault chassis; resilient suspension; hill-climbing qualities, 6 cylinders cast in 2 pairs of 3; side by side valves, etc.

118. Bafgh "Commercial Road Vehicles on Test." *Modern Transport*, V. 21, No. 540, July 20, 1929, pp. 18-19. Commer 30 cwt. (1½ ton) chassis designed for light transport, given full load road tests, acceleration, gear changing, braking, hill climbing, technical description of chassis.

119. Bd "Can Front Wheel Drive Aid Trucks?" P. M. Heldt. *Commercial Car Journal*, V. 37, No. 4, June 1929, pp. 18-19, 48 and 57. Claims that front-wheel-drive vehicle is less given to skidding are taken up.

120. Bacdg "Starting and Skidding of Automobiles." F. Raout. *Pratique des Industries Mecaniques,* V. 12, No. 4, July, 1929, pp. 167-169. Relation existing between force at rim, adhesion, and resistance to traction is discussed; what skidding consists of; practical means of increasing adhesion by increasing coefficient of friction, and by increasing loads supported by moving wheels.

121. ABbcj "Elements of Tractive Resistance Analyzed in Automotive Tests." L. E. Crooks. *Auto. Industries,* V. 60, No. 24, June 15, 1929, pp. 915-916. Results of 4 yrs. testing of automobiles, their air-resistance and coefficient of friction between tires and road surfaces, made upon highways in vicinity of Iowa Engr. and Exp. Stat.; forces opposing vehicular progress are increased 50 per cent by light gravel road surfaces and total 700 lb. per ton in deep mud; discussion of Iowa State College—Bull. No. 88, by T. R. Agg.

122. ABabcj "Tractive Resistance and Related Characteristics of Roadway Surfaces." T. R. Agg. Iowa State College Bull. No. 67, Feb. 6, 1924.

123. Bjklm "Air Resistance of Automobiles." E. H. Lockwood. *American Highways,* V. 8, No. 2, Apr. 1929, pp. 1-4. Results of four series of air resistance measurements carried out during past 8 yrs. in Germany and United States; rolling resistance and air resistance of sedans, coupés, roadsters, and touring cars of various makes on different grades.

124. ABbh "Light Truck Tractors for Oil Deliveries Given Strenuous Road Tests." R. B. Stafford. *National Petroleum News,* V. 21, No. 17, Apr. 24, 1929, pp. 32-34. Notes on tests of six makes of tank truck tractors at Gates Mills, near Cleveland, Ohio; tabulation of specifications and of results achieved on two hills and two types of road, along measured courses whose grades were determined; tests were conducted to show petroleum industry what could be expected of each type in actual service.

125. Ba "Bearing of Automobile Highways." J. Petier. Société Industrielle de l'Est. Bull. No. 192, Jan., Feb., Mar., 1929, pp. 36-41. General study of mechanics of motion of vehicles over highway; utilization of power, adherence, and stability of vehicles, vibration, critical velocities, shimmying, and pseudo-shimmying; progress in comfort, safety, and elimination of vibrations.

126. Bam "Influence of Form of Body on Speed and Fuel Consumption." M. Convierne. *Vie Automobile,* V. 26, No. 940, Feb. 25, 1930, pp. 65-68. Theoretical discussion and curves for calculation of required driving power, from weight, speed, coupling, and coefficient of air penetration.

127. ABabcj "Practical Tractive Ability Methods." A. M. Wolf. S.A.E. *Journal,* V. 27, No. 6, Dec., 1930, pp. 655-664. Importance of engine torque in relation to tangential force exerted by driving wheels at their points of contact with the road; typical tractive factors of modern motor trucks develop economic factor mathematically and thus be prepared to discuss tractive resistance as opposed to tractive effort; diagrams and charts illustrate air resistance and effect of different tires, and roadways.

128. Bm "Wind Resistance of Automoblies." F. N. Pawlowski. S.A.E. *Journal,* V. 27, No. 1, July, 1930, pp. 5-13 and (discussion) p. 14. Principles of aerodynamics and air flow around various shapes; coefficient of resistance to motion of streamlined body is approximately 1/30 that of flat plate having same frontal area; various methods of determining air resistance of automobiles, graphs and sketches illustrate test methods and results.

129. Dad "Effect of the Center of Gravity on Skidding Tendencies is Studied." J. Bradley and S. A. Wood. *Auto. Industries,* V. 63, No. 3, Dec. 6, 1930, pp. 833-834. Effect of moment of inertia about c.g.; effect of longitudinal position of c.g.; effect of height of c.g.; effect of transverse position of c.g. Abstract of paper read before Inst. of Auto. Engrs.

130. BCDad "The Skidding of Road Vehicles." *Engineering,* V. 130, No. 3388, Dec. 19, 1930, pp. 777-78. Editorial review of two papers read before Institute of Automobile Engineers, "Experiments on Factors Affecting Motion of 4-Wheeled Vehicle when some of its Wheels are Locked," and "Factors Affecting Behavior of Rubber-Tired Wheels on Road Surfaces," J. Bradley and R. F. Allen. (P. 149, E. Ind. 1930.)

131. Baf "Possibilities of Front Wheel Drive." *Commercial Motor,* V. 51, No. 1325, Aug. 5, 1930, pp. 884-6. How distribution of load and position of center of gravity affect wheel grip, acceleration, and retardation; difficulties of universal joints; larger amount of passenger space.

132. Bah "Front-Wheel Drive for Commercial Vehicles." Manlik. *Automobiltechnische Zeit,* V. 33, No. 3, Aug. 20, 1939. pp. 552-6. Investigation of traction and acceleration for various gradients show disadvantages of front-wheel drive for commercial vehicles.

133. Bd "The Skidding Danger." *Motor Transport,* V. 50, No. 295. Jan. 6, 1930, pp. 9-11. Various non-skidding devices for motor truck; success of chains in Switzerland; Parsons chains; Wheway anti-skids; Sheppee non-skids; Gilco non-skid chains.

134. Bad "How Do 6-Wheel Vehicles Affect Tire Performance?" G. M. Sprawls. *Power Wagon,* V. 44, No. 301, Jan. 1930, pp. 40, 45-8. Includes discussion of skidding and traction, among other things.

135. Ba "Automotive Engineer's Pocket Book." H. K. Thomas. 1931, pp. 16-20. Horsepower calculations and air resistance coefficient and formula.

139. Cbck "Tire Factor in Auto Riding Quality." R. W. Brown. *Rubber Age,* (N. Y.) V. 29, No. 8, July 25, 1931, pp. 361-63. Methods for measuring characteristics of tires which affect riding qualities, co-ordination between service performance and laboratory tests; tread design and tire traction.

140. Bm "Wind Tunnel Tests and Body Design." J. Andrade. S.A.E. *Journal,* V. 29, No. 1. July, 1931, pp. 29-32 and 56, 8 figs. (See also *Auto. Engr.,* V. 21, No. 287, Nov. 1931, pp. 542-544, 7 figs.) Wind tunnel tests of auto models, as made at U. of Detroit; general body and fender design to reduce air resistance and produce good appearance; methods of making tests so as to include ground effect and determine air flow pattern around car.

141. Bm "Future Auto Design Will be Influenced by Aerodynamics." A. Northup. *Auto. Industries,* V. 64, No. 24, June 13, 1931, pp. 908-909. Advantages of streamlining body contours with particular regard to appearance and efficiency.

142. Baim "Streamlining Applied to Automobiles." O. K. Marti. S.A.E. *Journal,* V. 29, No. 2, Aug., 1931, pp. 126-128 and 163. Results of wind tunnel tests of streamlined models show reduction of almost one half in wind resistance at speeds of 40 and 50 m.p.h., as compared with conventional American sedan model; theoretical analysis of horsepower required to maintain given car speed.

143. Bim "Advantage of Streamlining Lies in Power Economy at High Speeds." P. M. Heldt. *Auto. Industries,* V. 64, No. 20, May 16, 1931, pp. 762-765, 5 figs. Principles of streamlining and calculation of air resistance.

144. Baim "Wind Resistance *vs.* Car Performance." L. Schwitzer. S.A.E. *Journal,* V. 28, No. 6, June, 1931, pp. 631-635 and 668. European investigations regarding disturbance of air stream by bodies of various shapes; power losses and power utilization in car designed by Rumpler in Germany and in two German racing cars; novel design features of streamlined cars.

145. Bm "Tear-Drop Car." W. T. Fishleigh. S.A.E. *Journal,* V. 29, No. 5, Nov., 1931, pp. 353-358 and (discussion, 359-362. Automobile is developed which has remarkable possibilities in matters of decreased wind resistance, fuel economy, riding comfort, and clear vision, in addition to striking beauty and grace; result of wind tunnel tests upon models of this design and of conventional sedan.

146. ABCDbcd "Factors Affecting Behavior of Rubber Tyred Wheels on Road Surfaces." J. Bradley and R. F. Allen. Inst. Auto. Engrs., *Proceedings,* V. 25, 1930-31, pp. 63-84 and (discussion) 85-92. See also *Auto. Engr.,* V. 22, No. 277, Feb. 1931, pp. 73-78. Investigation of skidding of vehicles on road surfaces at Nat. Physical Lab.; devices for recording and analyzing forces set up in operation of tires with different types of treads on various road surfaces; sidesway force and braking force, coefficients as criteria of resistance to skidding.

147. Bm "Reo Royale Bodies Tested in Wind Tunnel at U. of Detroit." *Auto. Industries,* V. 64, No. 3, Jan. 17, 1931, pp. 96-97. Arrangement of test on models; power needed to overcome air resistance at speed of 80 m.p.h. is 12 per cent less than that required with previous type of body.

148. Ba "Four-Wheel Drive Will Improve Get-away of Today's High Powered Cars." P. M. Heldt. *Auto. Industries,* V. 64, No. 14, Apr. 4, 1931, pp. 538-540 and 546. Effect of weight distribution on tractive power, three types of differentials which can be used to distribute torque between front and rear wheels in proportion to their respective pressures on the road.

149. Ba "Practical Tractive-Ability Methods." A. M. Wolf. S.A.E. *Journal,* V. 28. No. 1. Jan. 1931, pp. 96-100. Discussion of Transportation Meeting Paper; concerned with desirability of calculating tractive ability from horsepower.

150. Bajh "Formulae for Rating Trucks." L. R. Buckendale. S.A.E. Journal, V. 29, No. 4, Oct., 1931, pp. 336-338, see also *Commercial Car Jl.,* V. 41, No. 5, July 1931, pp. 14-16 and 46; and *Petroleum World,* V. 28, No. 7, July 1931, pp. 53 and 55. Interpretation of formula taking into consideration gross vehicle weight, engine torque, total gear reduction, overall mechanical efficiency, radius of tire under load, gradient, and rolling resistance.

151. BCDd "Some Experiments on Factors Affecting Motion of Four-Wheeled Vehicle When Some of its Wheels are Locked." J. Bradley and S. A. Wood. Inst. Auto. Engrs., *Proceedings,* V. 25, 1930-31, pp. 46-62, see also *Auto. Engr.,* V. 21, No. 276, Jan. 1931, pp. 34-38. Program of Road Research at National Physical Lab. on behalf of Ministry of Transport; skidding characteristics as affected by weight distribution on wheels; height of c.g.; moment of inertia about vertical axis through c.g.

152. Bm "Elimination of Cowl Probable, Says Ralph Roberts, Briggs Designer." A. F. Denham. *Auto. Industries,* V. 66, No. 7, Feb. 13, 1932, pp. 224-7. Possibilities of streamlining illustrated by 2 examples.

153. Bim "Economy of Streamlining the Automobile." S.A.E. *Journal,* V. 30, No. 3, Mar. 1932, pp. 150-2. Investigation of air resistance in wind tunnels shows that from 60 to 84 per cent of power required to propel closed car is necessary to overcome air resistance at speeds of 40 to 80 m.p.h.; power savings by suitable streamlining of automobiles, amounting to 50 per cent at 40 m.p.h. and 65 per cent at 70 m.p.h.

154. Bim "Drag of Cars Charted in Wind Tunnel Tests." W. E. Lay. *Auto. Industries,* V. 66, No. 14, Apr. 2, 1932, pp. 520-2. Investigation of aerodynamic characteristics of simple body forms in wind tunnel of Aero. Eng., U. of M.; graphs illustrate value of air resistance coefficient as affected by shape of front and rear end of car.

155. Ba "Report of Project Committee on Tractive Resistance and Allied Problems." W. E. Lay. *Proceedings,* Highway Research Board, V. 12. pp. 66-75.

156. Bm "The Air Resistance of Motor Vehicles." W. E. Lay. Nat. Research Council Pamphlet, 1932. (Wind tunnel models.)

157. Bam "Development of Rear-Engine, Streamline Car." D. Burney. S.A.E. *Journal,* V. 30, No. 2 and 3, Feb. 1932, pp. 57-64 and (discussion) Mar., pp. 116-19. Reference to unconventional designs produced in Europe and America, such as front wheel drive and rear engined cars; elimination of nearly ½ air resistance of orthodox sedan and much steadier riding of high speed; analysis of forces and motions induced in car by location of c.g. and distribution of weight.

158. Cak "Rolling Resistance of Rubber Tires." E. H. Lockwood. *India Rubber World,* V. 27. No. 1, Oct. 1, 1932, pp. 13-15, 4 figs. Notes on rear-wheel dynamometer; measuring tire resistance; solid tire tests; cord and fabric-tire tests; loss of power in tires.

159. Bm "Aerodynamic Characteristics of Automobile Models." R. H. Heald. *Journal of Research,* National Bureau of Standards, Aug., 1933.

160. ABbd "Why Cars Skid." *Motor* (London), V. 65, No. 1670, Dec. 26, 1933, pp. 1030-1. How skid commences and develops and ways in which it can be corrected; dangerous road surfaces.

161. Bim "Is 50 Miles per Gallon Possible with Correct Streamlining?" W. E. Lay. S.A.E. *Journal,* V. 32, No. 4 and 5, Apr., 1933, pp. 144-56 and May, pp. 177-86; see also Editorial Remarks in *Engineering,* V. 135, No. 3513, May 12, 1933, pp. 521-2. April: Tables and Charts illustrating results of mathematical analysis and wind tunnel tests of variety of body designs. May: Study of test methods used; better fuel mileage with proper gear ratios; drawbar dynamometer for measuring air force on "floating envelope," fuel consumption at most efficient engine operation, suggestions offered to car builders.

162. Bm "Differences Between Wind-Tunnel and Road Conditions." S.A.E. *Journal,* V. 33, No. 2, Aug. 1933, pp. 261-7. Comments on paper by W. E. Lay (indexed from issues Apr. and May 1933) including: "Potent Factors Neglected in Many Streamline Tests," M. C. Horine; "Constructive Questions on Important Features," P. Altman; "Car-Resistance Reduction and Skin Friction Drag," H. G. Winter; "Dust Swirl Fuel Wastage Challenges Car Designers,"

E. G. Reid; "Wind-Tunnel Results Need Kid-Glove Handling," R. H. Upson; "Lay Amplifies Test Method Data and Answers Critics," W. E. Lay.

163. Bm "Side Winds Abate Performance Gains Hoped for from Streamlining." R. H. Heald. S.A.E. *Journal,* V. 33, No. 5, Nov. 1933, pp. 18-21. Data submitted are not considered to be applicable to all types of bodies, but are useful in indicating order of magnitude of lateral and longitudinal forces which arise as result of action of natural wind against moving automobiles.

164. Bm "Streamlining—Latest Researches." M. Platt. *Motor* (London). V. 63, No. 1645, July 4, 1933, pp. 903-6. Study of methods available for reduction of air resistance; main lines of attack on air resistance summarized as streamlining, smoothing and slimming; economizing power expenditure; selecting top gear ratio; influence of cross winds; streamlining reduces noise.

165. Ca "Future for Low-Pressure Tires Gradually Grows Brighter." A. F. Denham. S.A.E. *Journal,* V. 32, No. 4, Apr. 1933, pp. 138-41, (discussion) 141-3. Factors of appearances, riding qualities, traction, mileage, steering, blow-outs, and costs as related to low-pressure tire problem; experiences recorded by individual companies and engineers during past year.

166. ABbcd "Further Coefficient of Friction Tests." Stinson and Roberts. *Proceedings,* Highway Research Board, V. 14. pp. 131-147, 1934. Covers effect of tire tread and inflation and road surface on rolling coefficient: 1. Vertical-fiber wire-cut lug brick roads, asphalt; 2. Snow conditions.

167. ABa "Motor Vehicle Power Requirements on Highway Grades." Moyer, R. A. *Proceedings,* Highway Research Board, V. 14, pp. 147-186, 1934. *Lab. tests* on rolling resistance, power and fuel consumption. *Road tests* on gas. consumption on various surfaces and grades, free wheeling tests; rolling, air and engine resistance; acceleration and tractive effort.

168. ABbdef "Further Skidding Tests With Particular Reference to Curves." Moyer, R. A. *Proceedings,* Highway Research Board, V. 14, pp. 123-130, 1934. Skidding and braking tests on snow and ice, coefficients given for use in highway design.

169. Bm "Reflection Plate Representing Ground." Stalker E. A. *Aero. Sciences Journal,* V. 1, No. 3, Jl. 1934, pp. 151-2. Use of plate upon which boundary layer is removed or energized so that it does not present drag to air stream; reference to author's tests on auto bodies at Univ. of Mich.

170. Bm "Belt Method of Representing Ground." Klemin, A. *Aero. Sciences Journal,* V. 1, No. 4, Oct. 1934, pp. 198-9. Discussion of paper by Stalker, No. 169.

171. Bm "'Moving Ground' Developed for Wind Tunnel Testing of Car Models." A. Klemin. *Auto. Industries,* V. 71, No. 5, Aug. 4, 1934, p. 140. Results obtained so far bear out following general theory: 1) Absolutely free air resistance is apt to be unduly high; 2) mirror method approaches correct results more closely, but may deviate in either direction from correct results; 3) fixed ground board, owing to boundary-layer formation, gives results far too low.

172. Bm "Comparison of Ground Plane and Image Methods for Representing Ground Effect in Tests on Vehicle Models." R. H. Heald. National Bureau of Standards, *Journal of Research,* V. 13, No. 6, Dec. 1934, pp. 863-70. In order to simulate full scale conditions in wind tunnel it is necessary to represent presence of ground; approximate representation of ground effect obtained by means of either fixed ground-plane method or image method; experimental comparison of two methods.

173. Bm "Air Forces and Yawing Moments for Three Automobile Models." R. H. Heald. National Bureau of Standards, *Journal of Research,* V. 13, No. 6, Dec. 1934, pp. 871-8. Lift, drag, cross-wind force and yawing moment for 3 models having substantially different body shapes measured in a wind tunnel; results of model investigation, extrapolated to full scale, indicate presence of rather large lift, lateral force, and yawning moment under certain conditions of car speed and natural wind speed.

174. ABabcd "Skidding Characteristics of Auto Tires on Roadway Surface, and Their Relation to Highway Safety." R. A. Moyer. Iowa State College Bull., No. 120, 1934.

175. ABabji "Tractive Resistance as Related to Roadway Surfaces and Motor Vehicle Operation." R. G. Paustian. Iowa State College Bull., No. 119, 1934.

176. BDam "Bremsen mit Luftwiderstand?" L. Handl. *Automobiltechnische Zeit,* V. 37, No. 18, Sept. 25, 1934, p. 406. Braking by means of air resistance; short article suggesting that in developing high-speed automobiles, principles of air resistance braking as used on airplanes may be applied to automobiles.

177. Bm "La Résistance de L'air à L'Automobile." M. Andreau. Societas de Ingenieurs de l'Automobile, *Journal,* V. 7, No. 2, Feb. 1934, pp. 2577-94. See also *Auto. Industries* V. 71, No. 3, 4, and 6, July 21, 1934, pp. 74-6, July 28, pp. 116-7, and Aug. 11, pp. 172-5. In author's opinion so-called streamlined bodies do not reduce air resistance materially; developments on fenders, running boards, and headlights seem much more promising; it is doubtful whether changes being made in body form come under head of streamlining and whether motivation is not largely commercial.

178. CDac "Brake Design." B. B. Bachman. *Auto. Engineer,* V. 24, No. 317, Mar. 1934, pp. 108-11. Analysis of certain modifying factors; coefficients of friction between ground and tire; effective weight; distribution of braking effort on front and rear wheels; space available for brakes; brake types, factors influencing arrangement of brakes; brake function, brake control mechanism, pedal pressure, brake rating.

179. Bam "Dynamics of Modern Auto." G. L. McCain. S.A.E. *Journal,* V. 35, Nos. 1, and 5, Jl. 1934, pp. 248-56 and discussion, Nov., pp. 429-30. Enumeration in simple form of some of the outstanding features of new automobile dynamics; analysis of streamlining; effects of redistribution of passengers and units; results of riding quality model tests and weight distribution; forces acting upon planetary overdrive. Bibliog. of streamlining included.

180. Bm "Grundlagen fuer Modellversuche an Fahrzeugen." O. Lutz. *Automobiltechnische Zeit,* V. 37, No. 8, Apr. 25, 1934, pp. 211-12. Fundamentals for model testing of automobiles; it is shown that Froude's Law should be applied.

181. Bm "Possibilities of Truck Streamlining for Work in Dairy Industry." *Food Industries,* V. 6, No. 2, Feb. 1934, pp. 80-1. First fully streamlined tank truck, Texaco, built by Diamond T to order for Texas Co.

182. Bm "Design of Public-Service Body Work." A. J. Romer. Inst. Auto. Engrs., *Journal,* V. 2, No. 8, May, 1934, pp. 10-28. Includes material on "minimum wind resistance."

183. Ca "Tire Tests." R. Paustian. *India Rubber World,* V. 90, No. 4, Jl. 1, 1934, pp. 33-4. Road tests which show behavior of pneumatic tires under operating conditions.

184. Ca "Automobile as Tyre Testing Instrument." A. H. Nellen. *Rubber Age* (London), V. 15, No. 3, May 1934, pp. 70-3. Some information accumulated during more than 10 years' intimate connection with road testing of pneumatic tires; purposes of test. From Vanderbilt News.

185. Bm "Praktische Stroemungsforschung an Kraftfahrzeugen." M. Schirmer. *Automobiltechnische Zeit,* V. 38, No. 7, Apr. 10, 1935, pp. 176-81. Practical wind tunnel tests of automobiles; relation between automobiles and model tests; possibilities of error; streamlining of racing cars; stability of streamlined cars, practical examples.

186. Bm "Farewell to the Horseless Carriage." E. G. Reid. S.A.E. *Journal,* V. 36, No. 5, May 1935, pp. 180-9. Critical study of possibilities of improved auto performance and economy by aerodynamic refinement by demonstrating analogy between motor-car air resistance and airplane parasite drag; example illustrates benefits of aerodynamic research in latter field and potentialities of similar work in former pointed out; author's reply. Also discussion of paper.

187. Bm "Aerodynamic Resistance." M. Gignereaux. (English translation), *Auto. Engineer,* V. 25, No. 328, Jan. 1935, pp. 33-5. Application of wind-tunnel testing to problem of aerodynamic resistance of vehicles; necessity of tests on reduced models; scheme of elliptical wind tunnel possible tests on reduced models; test results in table and curves; examples.

188. . ABCabc "Der Kraftschluss zwischen Rad und Fahrbahn." Schuster and Weichsler. *Automobiltechnische Zeit,* V. 38, No. 20, Oct. 25, 1935, pp. 499-504. Contact between pneumatic-tired wheel and road, and its effect on tire; tests show that range of adherence force, so-called skid resistance should be regarded as due to expansion or shrinkage of rubber to fit small irregularities in road surface, and *not* to sliding friction.

189. Bam "Air Resistance and the Automobile." M. Andreau. *Auto. Engineer,* V. 25, No. 329, 330 and 331, Feb. 1935, pp. 75-7, Mar., pp. 113-4, and Apr., pp. 147-9. Interesting information obtained from wind tunnel and practical tests; results of tests, drag, illustrations, and tables.

190. Cabc "Tyre Factors in Vehicle Control." F. G. W. King. *Engineering,* V. 140, No. 3642, Nov. 1, 1935, pp. 367-70. Study of tire behavior and part played in vehicle control, with special ref. to anti-skid properties; result of tests made by Dunlop Rubber Co.

191. Bam "Wind Tunnel Tests Show Reduction in Resistance Obtainable with Rear-Engine Streamlined Model." E. G. Reid. *Auto. Industries,* V. 72, No. 3, Jan. 19, 1935, pp. 80-1. Research work carried out at Guggenheim Aero. Lab., Stanford Univ.; tests made on 4 models of streamlined cars; power requirements of conventional and streamlined sedans shown in curves.

192. Ca "Slip Angle, Camber, Load, Speed, Pressure, Rim Width, Size—How They Affect Tire 'Cornering' Power." *Auto. Industries,* V. 72, No. 3, Jan. 19, 1935, pp. 76-7. Performance data in curves.

193. Bm "Representation of Ground in Wind Tunnel." M. Ore. *Journal Aero. Sciences,* V. 3, No. 2, Nov. 1935, pp. 40-2. Experimental study of methods of representing ground in wind tunnel work by stationary plate, moving belt and image model methods; image model method gives flow pattern more closely corresponding to moving model than stationary plate method.

194. Bmi "Economics of Streamlining in Heavy Transportation." (Abstract L. H. Brown), S.A.E. *Journal,* 39: 516, Dec., 1936.

195. BDf "Piston Friction." Taylor. S.A.E. *Transactions,* V. 31, 1936, 8200-5.

196. ABdf "Speed *vs.* Safety on Straightaways; Skidding and Braking Tests." R. A. Moyer. Ill. diags., *Civil Engineering,* 6: 801-4, Dec., 1936.

197. ABbdef "Speed *vs.* Safety on Curves." R. A. Moyer. Ill. diags., *Civil Engineering,* 6: 801-4, Dec., 1936.

198. BDafm "Kuerzere Bremswege durch Erhoeherung des Luftwiderstandes." S. Sztatecsny. *Automobiltechnische Zeit,* V. 39, No. 15, Aug. 10, 1936, pp. 382-6. Decrease of brake stopping distance by increasing air resistance; theoretical math. analysis in order to determine whether, at high speed made possible by extensive speedway construction program in Germany, increase of air resistance of car during braking period may be applied advantageously.

199. Bm "Air Resistance and Air Resistance—H.P. Chart." P. M. Heldt. *Auto. Industries,* V. 75, No. 12, Sept. 19, 1936, p. 379. Chart determines air resistance encountered by car at any given speed; example of application.

200. Bm "Atmospheric Resistance of Motor Vehicles, Tests of 1932." Conrad and Dawley. Kansas State College, Eng. Exp. Station, Bull. No. 33, V. 20, No. 1, Jan. 1, 1936, 59 p. Wind tunnel determination of wind resistance of 54 autos together with independent check on two cars made by coasting tests presented; results of investigations presented as evidence, not as proof; these cars represent 14 makes including 7 yearly models and 4 body types.

201. Bm "Luftevideratand von Kraftfahrzengen-Modellversuche und Wirklichkeit." C. Schmid. *Automobiltechnische Zeit,* V. 39, No. 17, Sept. 10, 1936, pp. 425-35. Air resistance of automobiles; report of series of tests given for comparison between model tests and actual values regarding air friction in travelling.

202. Bm "Windkanalmessungen an Omnibusmodellen." *Automobiltechnische Zeit,* V. 39, No. 6, Mar. 25, 1936, pp. 143-50. Wind tunnel measurements on motor bus models; air resistance characteristics of various types studied by aid of three component scales; measured air forces served for valuation of results.

203. BDdf "Jacknifing in the Operation of Tractor-Trailer Combinations Analyzed." P. M. Heldt. *Auto. Industries,* 75: 739-41, 759, Nov. 28, 1936.

204. ACbd "Skid Tests on Bridge Floors." Ill. *Roads and Streets,* 80: 56, D '37.

205. Ca "20,000 Tire Tests." A. F. Denham. Ill. *Auto. Industries,* 77: 876-7, Dec. 18, 1937.

206. Ca "Determination of Variation in Unit Pressure Over the Contact Area of Tires." L. W. Teller and J. A. Buchanan. Ill. diags., *Public Roads,* 18: 195-8, Dec. 1937.

207. ABai "Effect of Highway Design on Vehicle Speed and Fuel Consumption." Oregon State Highway Department, 115 p., bibliog. (p. 102), Salem, 1937.

208. BDf "Engine Friction; Degree of Error in Relation to the Motoring Tests." M. K. McLeod. *Auto. Engineer,* bibliog., 27: 57-8, Feb. 1937.

209. Bm "Aircraft Trend in Body Design from the Aircraft Viewpoint." R. H. Upson. S.A.E. *Journal*, 41: 291-2, July, 1937.

210. BDf "Friction of Compression Ignition Engines." C. S. Moore. *Auto. Industries*, 76: 885-6, June 12, 1937.

211. Bmi "Advanced Streamlining Shows Operating Savings; New Tractor-Trailer." Ill. *Food Ind.*, 9: 522-35, 1937.

212. Bm "Aerodynamic Shrouding for Vehicles on the French State Railways." Ill. diags., *Engineer*, 164: 209-12, Aug. 20, 1937.

213. Bm "Peugeot Streamlined Body." Ill., *Auto. Industries*, 76: 766, May 22, 1937.

214. Bm "Streamlined Vehicle." *Machine Design*, V. 9, No. 6, June 1937, p. 45.

215. Bm Data From Scale Models in Wind Tunnel and from Deceleration Tests, Stuttgart Technical College undertakes research. *Auto. Industries*, 77: 651-2, Oct. 30. 1937.

216. ABai "Effect of Highway Design on Vehicle Speed and Fuel Consumption Studied in Oregon." J. Beakey. *Public Roads*, 18: 117-18, Aug. 1937.

217. Aabc "Surface Friction of Roads." *Engineering*, 143: 15-16, Jan. 1, 1937.

218. Abcd "Road Surface Resistance to Skidding." *Roads and Streets*, 80: 39, Dec., 1937.

219. Bd "Chained to Safety" (Use of anti-skid tire chains.) A. C. Carruthers. Ill., *Safety Eng.*, 74: 34-5, Nov. 1937.

220. ACabc "L'envirage et la tenue de la route." M. Julien. Société des Ingenieurs de l'Automobile, J. V. 10, No. 4, Apr. 1937, pp. 252-9. Article discusses skidding and adherence of automobile tires on road; contact between tire and road during rolling of wheel; elementary theory of contact giving short methematical analysis.

221. BDf "Der Verbrennungsmotor als Bremse." L. Richter. *Automobiltechnische Zeit*, V. 40, No. 13, July 10, 1937, pp. 325-9. Internal combustion engine acting as brake; notes on braking action of engine with and without auxiliary equipment.

222. Bd "Elimination of Skidding." (Buses.) *Transit Journal*, 81: 166, May, 1937.

223. Bim "Advanced Streamlining Shows Operating Saving, New Tractor-Trailers." Ill. *Food Ind.*, 9: 522-3, Sept. 1937.

224. Da "Braking Efficiency." S. Reis. *Auto. Industries*, 76: 422, Mar. 13, 1937.

225. Da "Differential Braking." *Auto. Engineer*, 27: 29, Jan. 1937.

226. Cab "Load Distribution over Tire-Ground Contact Area." Diags., *Auto. Industries*, 77: 328-354, 1937.

227. Bam "Aircraft and Automobile Manufacturers Test New Models at the Univ. of Detroit Laboratories." J. Geschelin. Ill. diags., *Auto. Industries*, 78: 660-2, May 14, 1938.

228. Ba "Outlook on Legal Performance Requirements (of Motor Trucks), Study of Speed on Hills." J. T. Thompson. Ill. diags., S.A.E. *Journal*, 42: 209-14; Discussion by W. F. Childs, Jr., May, 1938.

229. Ca "Physics of Rubber, as Related to the Automobile." W. F. Busse. Ill. diag., *India Rubber World*, 98: 42-5, Sept., 1938.

230. Bf "Braking Efficiency Measured by New Device." *Safety Eng.*, 76: 38, Aug., 1938. Braking, reaction time, etc.

231. ABbcd "British Studies of Skidding." 3rd Annual Report of the Road Research Board, Roads and Streets, 81: 66, Feb., 1938.

232. Cbcd "Research Makes Better Tires." D. S. Tuttle. Ill. *Commerce and Finance*, 27: 129-30, Apr. 1, 1938. Goodrich overcomes skidding.

233. Bd "Skid Danger Increases with Speed." Ill. *Scientific American*, 158: 227, Apr., 1938.

234. Cb "Rational Approach to Tire Tread Design." J. M. Lessells. Ill. diags., *Mechanical Engineer*, 61: 125-30, Feb., 1939.

235. ABh "Comparison of Methods for Determining the Hill-Climbing Ability of Trucks." C. C. Saal. Ill. *Public Roads,* 19: 233-9, Feb., 1939.

236. ABj "Tractive Resistance of a Motor Delivery Wagon on Different Roads and at Different Speeds." A. E. Kennelly and O. R. Schurig. AIEE Annual Convention.

237. Ca "Power Losses in Automobile Tires." W. L. Holt. National Bureau of Standards, 1922, No. 213.

238. Bm "Aerodynamic Characteristics of Auto Models." R. H. Heald. National Bureau of Standards, 1933, no. 2.

239. Bm "The Development of a Rear-Engine Streamlined Car." Sir Dennistoun Burney. S.A.E. *Transactions,* V. 27, p. 57.

240. Baim "Economy of Streamlining the Automobile." O. G. Tietjens. S.A.E. *Transactions,* V. 27, p. 150.

241. Bm See also: "Air Resistance of High Speed Automobiles." A. Persu, p. 25-27, discussing future design of streamline autos.

243. Bm "Air Resistance of Automobiles." Eshbach. Handbook of Engineering Fundamentals, 9-69.

244. ABajkml "Michigan State Highway Department Investigations of Truck Performance on Grades." Burton, Lay, and Rogers.

MOTOR VEHICLE OPERATING COSTS AND RELATED CHARACTERISTICS ON UNTREATED GRAVEL AND PORTLAND CEMENT CONCRETE ROAD SURFACES

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SYNOPSIS

This is a progress report on a cooperative project between the Iowa Engineering Experiment Station and the Public Roads Administration for the purpose of determining the effects of roadway type and condition upon the operating costs of motor vehicles.

Five new automobiles are being used, two on gravel, two on concrete, and the other on both surfaces. Complete and detailed records are kept on consumption rates of fuel, oil, tires, etc., costs of all items, measurement of wear and other data. As the project has been in operation for only a year, sufficient data are not yet available for final conclusions. However, certain trends seem to be definitely indicated.

After 36,000 miles of driving of each car, the average gasoline mileage for the first year for cars driven only on concrete is about 2 miles per gallon greater than on gravel, and the gravel cars use about twice as much oil. Tire wear on gravel was practically double that on concrete at 25 m.p.h. and at 50 m.p.h. was 50 per cent greater. Punctures were in the ratio of about 58 to 1 for gravel over concrete. Considerably more wear has been evidenced on brake drums, cylinders, and bearings in the gravel cars. Total operating costs on gravel averaged about ½ cent per mile higher than on concrete for the first year. Tests made with a riding comfort meter indicated that the smoothest gravel section in the test was rougher at 40 m.p.h. than the roughest concrete at the same speed.

This project was established for the purpose of determining the effect of roadway type and condition upon the operating cost of motor vehicles and upon safety, comfort, and ease of driving as observed in road tests primarily designed to measure operating costs. It was organized by the Iowa Engineering Experiment Station, Dean T. R. Agg, Director, as a cooperative project with the Public Roads Administration, Washington, D. C., during May, 1938. The writer is in direct charge of the project and is assisted by Robley Winfrey of the Experiment Station staff. Seven driver-observers and two part time research assistants,* recent graduates in the civil, mechanical, electrical, and general engineering fields, have assisted in developing test equipment and have conducted the special tests and the routine driving tests.

An advisory board, consisting of nine members made up of representatives of automotive, rubber, and highway organizations, was invited by the Public Roads Administration to meet periodically with the active workers on the project as technical advisors. Three consultants with wide experience in fleet and test car operation were invited by the Experiment Station, also, to serve as technical advisors.

Active work on the project started July 1, 1938. Five test cars of the same make and model were purchased in June and were delivered by July 1. It was decided to operate two of the test cars exclusively on portland cement concrete and two on untreated gravel. The fifth car is a general purpose test car. During the first three months all of the preliminary development work was carried out including breaking in the test cars. Routine tests were started in October,

* Driver-observers:
Myron Berry
Jack Clapsaddle
Keith Cochran
Herman Klemesrud
Gustave Scheffler
Clarence Schnare
Glenn Tesdall
Research assistants:
Adrian Legault
Robert Sawyer

1938, and have been carried on continuously since that time.

In this report of progress, a summary of the important data obtained during the first year of routine operation on the two road surfaces involving 32,000 miles of travel per car is given. The data and figures are being presented with only such comments and interpretations as it is felt may be necessary to clarify them or indicate the advisability of considering changes or additions in the future work of the project. While certain definite trends in the various items of operation are now evident it is felt that another year involving about 30,000 miles of travel per car will be necessary before reasonably complete and accurate data in all the items of cost and operation can be reported and before valid conclusions and interpretations may be reached. The report is offered at this time to inform and interest highway engineers and others in this important undertaking and to receive suggestions for improving the future work on the project.

Objectives

The main objective of this investigation is to determine the influence of the type of roadway and its condition upon the operating cost of motor vehicles by driving tests. The project is being conducted so that the differentials in each item of operating cost due to roadway type and condition are, in so far as they can be measured, accurately determined upon a basis of year-round travel.

A second objective is the determination of operating characteristics of motor vehicles on the various surfaces and the recording of observations which are related to safety, comfort, and ease of driving on these surfaces which can conveniently be made in conducting road tests primarily designed to measure operating costs. These are the intangibles in road cost comparisons reflected in the cost of highway accidents and the value of driving satisfaction which cannot be definitely evaluated but which the highway engineer cannot disregard in planning highway programs.

Need for Investigation

There is need for knowing accurately the cost differentials in motor vehicle operation which are due to roadway type and condition in connection with the planning of programs for the construction, maintenance, and extension of improved roads throughout the country, and in relocation of roads to save distance, to relieve congestion, and to increase the safety and ease of driving.

On state highway systems of the country with a total mileage of 465,237 reported on January 1, 1939,[1] there were 126,507 miles reported as untreated earth, sand clay, gravel, and macadam roads. Since accurate operating cost data are not now available for these surfaces, the economic justification of the extent of future improvements on them is in doubt. In the secondary and local road systems of the country there are probably 600,000 miles of similar surfaces carrying sufficient traffic to make the cost of operation an important, if not the most important consideration in determining the nature of future improvement and maintenance. On roads in Iowa and similar mid-western states where traffic exceeds 50 or 100 vehicles a day, the highway costs rarely exceed ½ cent per vehicle mile. Recent studies made under the writer's direction of operating costs of cars driven by rural mail carriers indicated that the average cost differentials due to the type and condition of the surface may easily be 1 to 2 cents per vehicle mile.[2] The significance of a

[1] *American Highways*, pp. 12-13, April, 1939.

[2] *Proceedings*, Highway Research Board, Vol. 17, p. 53, 1937, Vol. 18, p. 44, 1938. Also, Bulletin 143, Iowa Engineering Experiment Station, 1939.

reduction of 1 cent per vehicle mile in operating costs may be better understood when expressed in annual costs. Thus, if an untreated gravel road carrying 500 vehicles per day were covered with a high type surface, a reduction in operating costs of 1 cent per vehicle mile would provide an assumed annual saving of $1,825 per mile of road. For other traffic volumes, savings would be in like proportion. On the basis of these large savings which would effect millions of car owners using untreated surfaces, it is evident

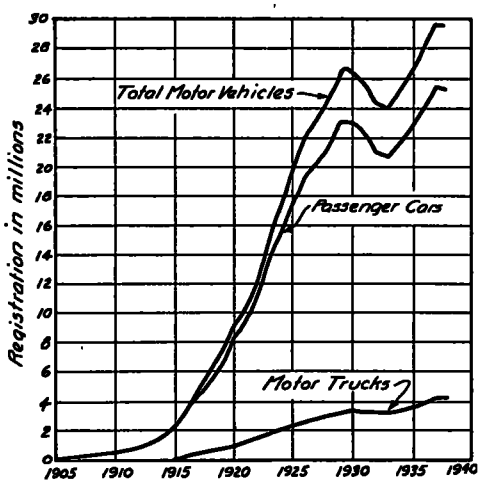


Figure 1. Motor Vehicle Registration in the United States. Based on report by Public Roads Administration.

that treating or paving these surfaces is justified.

Also, there is need at this time for a comprehensive study based on road tests of the many factors which relate to all phases of motor vehicle operations involving both costs and safety, comfort and ease of driving motor vehicles. The phenomenal increase in motor vehicle registration in the United States during the past 20 years (Fig. 1) should be reassuring to engineers that there is great demand and necessity for this form of transportation. At the same time the obligation to provide transportation at the lowest cost should be recognized. It should be evident that except for the steady increase in driving speeds and the decrease in car costs since 1900 as shown in Figure 2, the phenomenal increase in car registration would not have been likely. Improvements in roads and in cars have made these changes possible.

The trend in the curve for car costs indicates that these costs are flattening out in recent years and that further reductions will be possible only from

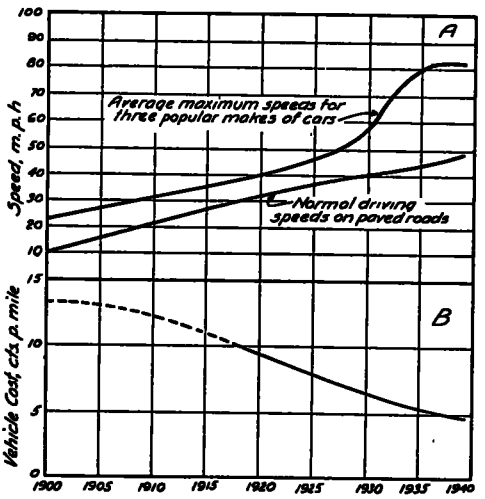


Figure 2.

A—Vehicle Speeds on Paved Roads in the United States. Based on tests by the Iowa Engineering Experiment Station and various references in the Literature.

B—Average Vehicle Costs on Road in the United States. Based on car cost records, Iowa Engineering Experiment Station.

improvements in cars and highways based on a careful investigation of every cost item as is contemplated in this study. Previous studies by the Iowa Engineering Experiment Station have revealed that the type of surface or its condition have considerable effect on fuel consumption (Fig. 3). These studies have been criticized in that the results are incomplete; they cover tests over

short test courses and usually for the summer season only. Differences in tire, oil, and maintenance costs due to the type and condition of the surface may be equally important. In the present study it is proposed to obtain a measure of all these cost items over long courses upon a year-round basis.

no special treatment and shipped to the local dealer in Ames by rail. All of the cars were equipped with 1939 Goodyear Allweather Tread (G40) first line 4-ply tires, furnished by the manufacturer from the same run of tread stock to assure uniformity in wear. The tires were built of standard cotton cord and

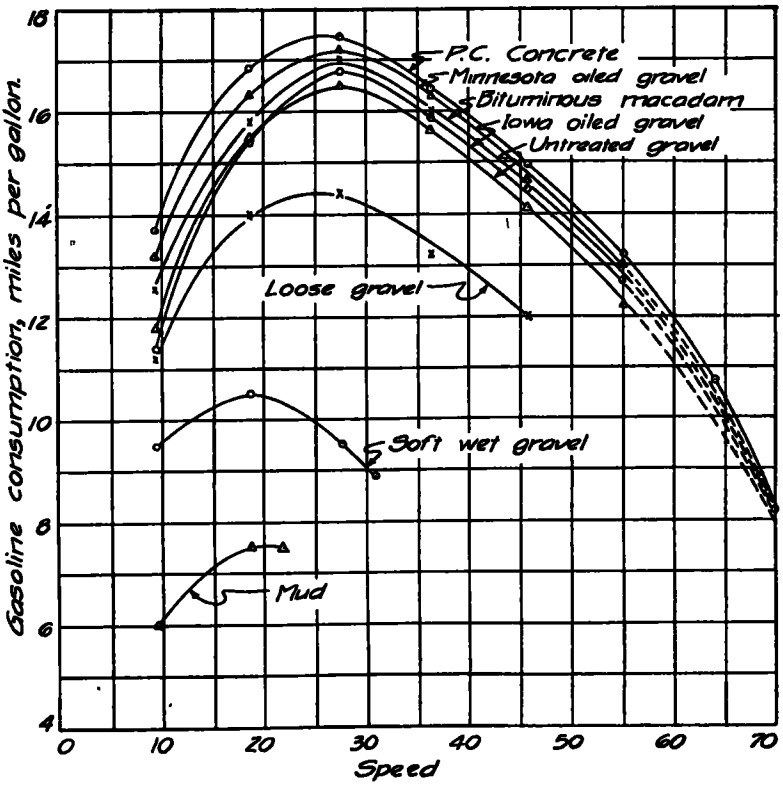


Figure 3

THE TESTING PROGRAM

Test Cars

The five test cars, 1938 Chevrolet Deluxe Town Sedans, are fairly representative of the popular priced cars which comprise the bulk of the passenger car traffic in this country. With the exception of the tires the cars were taken at random from the assembly line with

are of the same grade furnished car manufacturers for original equipment on new cars. The uniformity of the tires has been demonstrated by the fact that five tires on the two gravel cars failed during the same week of operation after a tire mileage of 26,400 plus or minus 300.

Test Routes

The concrete pavement test route is 232.8 miles long located in central Iowa,

and closely paralleling it is the gravel route 237.0 miles long (Fig. 4). The concrete and gravel surfaces were selected for this investigation because they represent important surface types for which the differential effects are large and are more easily measured than if concrete and one of the treated types were compared. Also, portland cement concrete surfaces provide a good standard of comparison because their surface characteristics are similar in every part of the country, especially after they have been

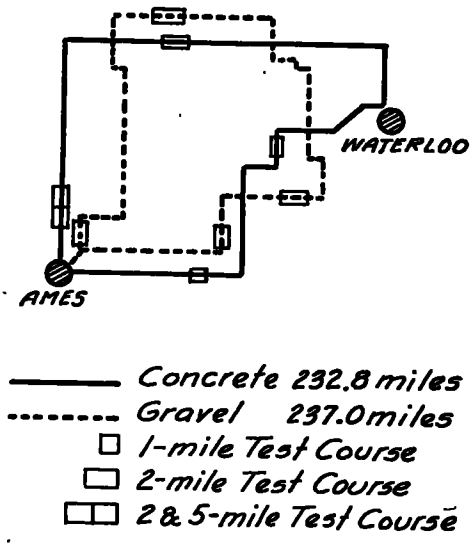


Figure 4. Test Routes and Test Courses

subjected to traffic for more than three years, as all of them have been on the test route used in this study.

The test routes were logged during the summer of 1938 with special attention to the number of stops, curves, grades, and length of each route. A final route on each surface was selected to hold to a minimum the differences which might be caused by stops, curves, and grades. Each route is divided into 22 special test sections which are identified with markers at the fence line. Through cities and towns the sections are confined to built up sections and are from 1 to 4 miles in length. In rural areas they are from 5 to 15 miles in length. These sections are used to measure variations in gasoline consumption and speed for straight level open sections, hilly sections, curve sections, and to determine effect of wind, temperature, etc. The effects of slow speeds, traffic interference, and stop and go driving are measured in towns and cities. Also, four measured test courses, of which all except one are two miles in length, have been established on each route on straight level sections suitable for daily calibrations of speedometers and for gasoline consumption measurements at the cruising speed for the day's run.

The greater part of the paving on the concrete route was laid during the last 10 years although in Marshall County the paving is 16 and 17 years old. A belt finish was used on practically all of the concrete which has been worn smooth by traffic for the greater part of the route.

The gravel roads are located largely on the county trunk system although 15 miles are in the Iowa primary road system and are maintained by the Iowa State Highway Commission. The roads are the feather edge type constructed by applying 500 to 1,500 cu. yd. (usually about 800 cu. yd.) of pit run gravel per mile on the graded earth road surface, and then letting traffic and the weather effects compact it. The upper two inches of these gravel surfaces are generally loose and lack clay binder as is shown in Table 1 giving laboratory analyses of samples taken from the gravel route and in Table 2 the weight of loose gravel per sq. ft. at various points on the route. The liquid limit, plastic index and the clay content are notably higher in the gravel course 2 to 4 in. below the surface than in the top 2 in. The method of maintenance used on these roads frequently results in a windrow of gravel

TABLE 1

RESULTS OF SOIL TESTS OF GRAVEL SAMPLES TAKEN FROM GRAVEL ROUTE DURING AUGUST, 1939

Miles clockwise from Ames laboratory	Upper 2 in. of surface						From 2 in. to 4 in. under surface					
	Liquid limit	Plastic index	Composition percent				Liquid limit	Plastic index	Composition percent			
			Clay 0-.005 mm.	Silt .005-No. 270	Fine agg. No. 270-No. 10	Coarse agg. No. 10-1½ in.			Clay 0.005-mm.	Silt .005-No. 270	Fine agg. No. 270-No. 10	Coarse agg. No. 10-1½ in.
4.7	23.5	6.9	11.11	8.18	52.40	28.31	22.5	5.0	11.02	9.11	55.81	24.06
11.0	22.5	3.1	7.98	7.13	43.94	40.95	31.8	20.1	20.89	24.05	48.76	6.32
54.0	23.5	0.0	10.81	7.65	64.27	17 27	27.2	7.4	15.34	19.18	52.00	13.48
98.3	13.5	0.0	8.80	2.35	66 88	21.97	18.8	0.0	9.88	8.97	55.40	25.75
116.3	20.0	3.3	1.92	24.19	39.04	34.85	26.5	5.4	14.18	15.12	49.00	21.70
135.0	16.9	0.0	11.80	7.41	61.86	18.93	23.3	5.8	22.35	13.90	50.32	13.43
156.0	26.2	6.2	12.49	15.66	43.88	27.97	39.2	13.7	24.20	34.46	18.46	22.88
190.0	18.5	0.0	8.37	7.80	50.10	33.73	32.5	13.2	17.24	25.47	28.59	28.70
201.7	20.4	6.5	9.52	10.12	44.33	36.03	30.0	7.3	16.89	35.11	20.65	27.35
211.9	21.2	4.1	10.86	11.56	39.78	37.80	32.8	14.2	23.68	25.42	43.51	7.39
217.8	21.0	3.0	11.80	16.41	22.15	49.64	42.3	12.3	23.03	20.78	29.18	26.91

NOTE: The road surface of approximately 65 out of a total of 237 miles on the gravel route is soft and muddy during wet weather.

TABLE 2

LOOSE GRAVEL ON GRAVEL ROUTE ROAD SURFACE

Miles clockwise from Ames laboratory	Weight of loose gravel lb. per sq. ft.			Miles clockwise from Ames laboratory	Weight of loose gravel lb. per sq. ft.			Miles clockwise from Ames laboratory	Weight of loose gravel lb. per sq. ft.		
	Maximum	Average	Minimum		Maximum	Average	Minimum		Maximum	Average	Minimum
5.0	2.86	0.90	0.10	91.3	14.00	4.28	1.25	171.8	2 85	1.22	0.50
10.0	14.45	3.91	0.94	102.3	34.95	12.65	3.75	183.2	14 70	3.91	0.40
15.0	8.71	3.76	0.20	107.9	4 75	1.90	0.80	190.2	17.75	6.26	0.26
18.0	5.35	3.76	2.18	115.5	5.60	2.75	1.05	195.1	19.00	6.06	0.87
25.0	15 05	4 43	0 34	123.3	3.10	1.73	0.90	200.1	7.07	2.52	Nil.
34.0	3.87	2.69	0.85	130.5	19.20	4.93	0.85	201.4	5.15	2.01	0.10
44.1	11.30	2.73	0.20	141.2	24.85	7.58	1.58	205.2	8.57	5.02	0.41
55.7	3 50	1.69	0 65	149.1	2.93	2.08	1.30	211.8	6.33	4.45	2.22
68.0	6.20	3.61	0.95	154 5	4.47	1.80	0.22	214.9	14.51	2.90	0.91
80.0	6.60	2.43	0.45	163.6	5.85	2.70	0.43	217.7	4.17	2.90	1.08
								220.8	4.69	2.13	0.84

NOTE: The maximum, average, and minimum weights are at points on one transverse section of surface at a representative location in the road for the given mileage.

along the center of the road or along the edge of the traveled way which may be 5 or 6 in. deep as indicated by the large maximum values for loose gravel at certain locations on the route (Table 2). On about 65 miles out of a total of 237 miles of gravel route, the gravel is so thin and the clay content so high that the cars form ruts and the roads are soft and

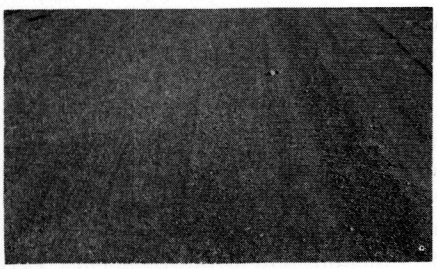


Figure 5. Typical view of gravel road, firm and smooth when dry



Figure 6. Typical view of a washboarded gravel road

slippery after a heavy rain. These sections are usually firm and smooth in dry weather (Fig. 5). The remaining miles of the gravel route are generally firm when dry or wet but with varying amounts of loose gravel as indicated in Table 2. In dry weather washboarding (Fig. 6) is common on the greater part of the gravel mileage and the roads are dusty (Fig. 7).

Preliminary Development Work with the Cars

After the cars were delivered by the local dealer, they were taken to the Iowa Highway Commission garage where an expert mechanic assigned to do all the servicing and mechanical work on the cars, checked them thoroughly and brought them up to the manufacturer's specifications for all adjustments and defects found. During the break-in

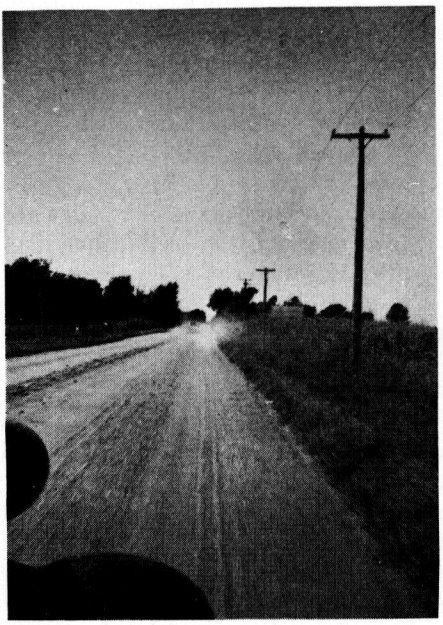


Figure 7. Typical view of a gravel road when dry showing dust raised by a car

period the cars were operated on a definite speed schedule in which speeds were raised from 25 m.p.h. to 75 m.p.h. in 5 m.p.h. increments every 300 miles over 3,000 miles of driving. The cars were used in logging the test routes and similar work during the break-in period.

For convenience in referring to them, the cars were numbered as follows:

Car No. 1 is the general purpose test car operated on both gravel and concrete routes.

Cars 2 and 3 are operated exclusively on the gravel route except for standard performance tests run on concrete.

Cars 4 and 5 are operated exclusively on the concrete route.

During the break-in period a valve spring broke in Car No. 2 at 698 miles with minor damage. But at 1,422 miles another valve spring broke on Car No. 2, dropped the valve onto the piston, punched a hole in it, and then went down into the crankshaft, doing considerable damage. The factory representative from Des Moines recommended a new block assembly, a new head, and a new pan and the necessary small parts to replace defective parts under the company's warranty agreement. This was done and the engine and car were then given a careful break-in to avoid extra miles introduced by miscellaneous driving with the other cars. This car ended up with 640 more car miles and 760 less engine miles than the corresponding mileages of the other cars at the end of the break-in period which is a small difference when compared with the 36,000 miles on each of the cars after the first year of routine tests.

After the cars had been driven 1,000 miles each during the break-in period, the pan of each car was dropped, and the engine head and wheels were removed for a careful inspection and for initial measurements to be used in determining cylinder wear, bearing wear, brake lining wear, and brake drum wear at stated intervals of driving, such as after the first year's operation. Standard adjustments to manufacturer's specifications were made to bring the cars to the same level of performance.

All of the equipment on the cars was standard except that upon recommendation of the Advisory Board, Cars No. 2 and No. 4 were equipped with an oil bath air cleaner and an oil filter. No changes were made in the equipment on Cars No. 3 and No. 5.

By the end of the break-in period the cars were all equipped with the various instruments, gauges, and measuring devices used in the special and the routine tests. After a final checkup at 3,200 miles by the mechanic the cars were ready for the standard performance tests. These consisted of maximum acceleration in all gears on a level concrete test course and deceleration or coasting tests in high gear and in neutral on the same course. Gasoline consumption was determined at each ten-mile speed from 10 to 70 m.p.h. over a level five-mile course, using shorter distances for lower speeds. The speedometers and odometers were checked over the measured five-mile course. By comparing results of these tests, the differences between cars were found to be so small that no correction factor for any item was deemed necessary.

Speed Surveys

During the summers of 1938 and 1939 speed surveys were made at various points on the test routes to determine the distribution of driving speeds to be used in the routine tests. The results of these surveys are shown in Figure 8. It is of interest to note the slight increase in speed on concrete and practically no change on gravel in the 1939 surveys. The average speed on the concrete in 1938 was 47½ m.p.h. and on the gravel route 38 m.p.h. In Figure 9 the speed distribution of passenger car traffic on gravel is shown in a form which indicates the wide variations in speeds on gravel due to road condition. The reduction in percentage of cars in the 35-40 m.p.h. group at certain locations indicates that some drivers apparently try to avoid the discomfort experienced on gravel roads at certain speeds by driving at a higher or lower speed.

Driving schedules have been worked out to provide a speed distribution for the test cars closely approximating the

average speed distribution for traffic on the test routes. For the winter season the speed distribution is changed to give lower average speeds on account of poor road conditions, especially on the gravel route. The mileage traveled by each test car during each season is adjusted to the relative seasonal mileage as indicated by the Iowa Highway Commission traffic counts made with the automatic traffic counters on roads in this vicinity. To maintain a uniform basis for comparison, the same seasonal mileage has been used for both the gravel and the concrete cars.

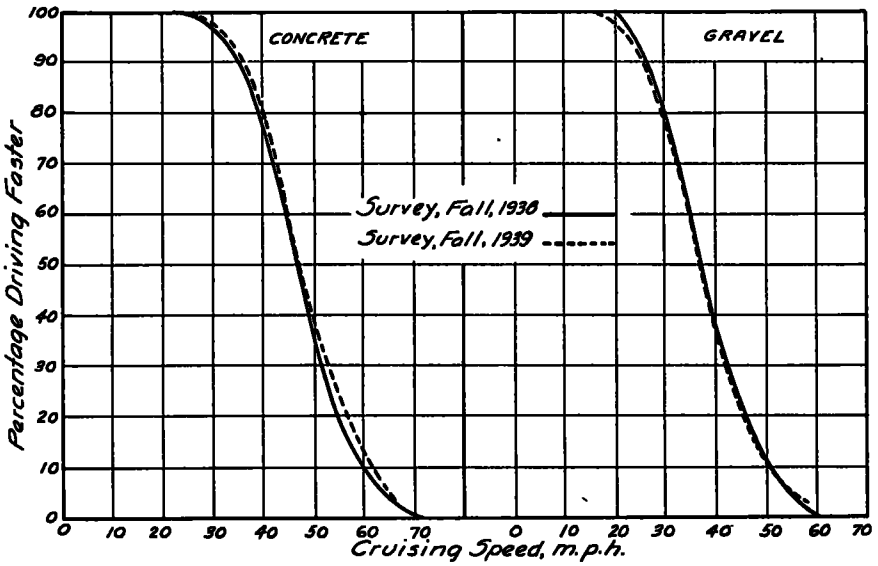


Figure 8. Observed Speed Distribution of Passenger Car Traffic on Concrete and Gravel Test Routes

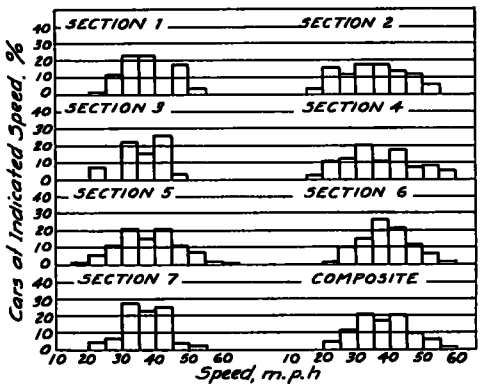


Figure 9. Observed speed distribution of passenger car traffic on gravel route, 1938, to show variations in the normal distribution as the decrease in the 35-40 m.p.h. speed due to washboarding and similar road conditions.

General Plan of Operation

The general plan of operation provides for operating the two gravel cars No. 2 and No. 3 on the gravel route one day and the two concrete cars No. 4 and No. 5 on the concrete test route the next day. The directions are alternated on each successive run. Four of the driver-observers (two to each car) are sent out each day with two cars. The men alternate driving every 60 miles and are rotated from car to car on successive days. The cars are run in tandem a block or more apart and alternate lead position every 60 miles. On the off days when the cars are not out on a run an opportunity is provided to service the cars,

measure the tires, or to give them such attention as may be necessary.

The practice of driving on alternate days on the two test routes has worked out quite well with every indication that weather effects are very nearly equalized over a season and for periods longer than a season. The greatest variations due to weather are those caused by changes in wind velocity. For the first three seasons the wind velocity on the days when the gravel cars were out averaged 7.8 m.p.h. as compared to 7.1 m.p.h. when the concrete cars were out, a difference so small as to have no practical significance.

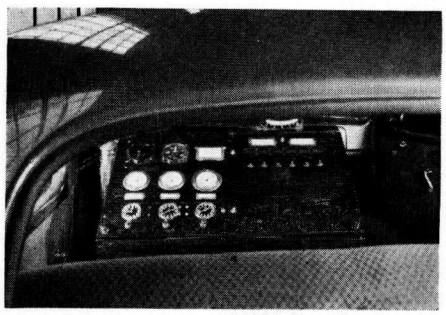


Figure 10. View of instrument panel from rear window of test car showing thermocouple switch, air speed indicator dial, electric counters, pressure gauges, and electric clocks.

Observed Data

Although it might be possible to obtain some measure of such cost items as gasoline, oil, tires and maintenance over large mileages without the extensive instrumentation developed for this project, little opportunity would then be provided to explain the results or to explain the effects of the many different driving conditions encountered during a given season or during the year. Accordingly every car was equipped with a number of gauges, meters, and measuring devices (Fig. 10) which the observers read at the beginning and end of each of the special sections on the route. In this way a complete record of every important operating condition known to effect the results or which is likely to be helpful when interpreting the results is obtained. Furthermore, these observations provide a check on the uniformity in the driving techniques of the drivers and provide the means whereby all the drivers will be aware of differences in their driving.

The observed data may be divided into three groups: (a) weather conditions; (b) driving conditions and vehicle performance; (c) roadway conditions and traffic interference.

(a) *Items recorded under weather conditions*

Air temperature
Wind direction and velocity
Humidity
Barometric pressure, and
Precipitation, if any, such as rain or snow

(b) *Items related to driving and vehicle performance*

Nominal speed
Actual average speed, based on actual running time and distance traveled
Number of stops
Engine hours
Fuel consumption
Oil consumption
Number of clutch and brake applications
Braking time
Engine revolutions
Temperatures by using thermocouples in:
  Crankcase
  Radiator inlet and outlet
  Intake manifold
  Exhaust manifold
  Air
  Carburetor intake
  Transmission
  Differential
  Gasoline meter
Pressures in exhaust line and fuel line, and the manifold vacuum
Tire wear by tread depth
Labor hours of servicing
Repair items
Labor hours for repair
Fuel-air ratio by exhaust analyzer
Steering effort and total steering angle movement
Wheel slippage
Braking force

(c) *Items related to the road surface and traffic*

Type and miles of each surface
Condition of each surface
Coefficient of friction of various sections
Rolling resistance
Roughness of surface as measured by recording accelerometer and road profilometer
Hardness or softness of surface as measured by impact penetrometer
Amount of loose material
Density and speed of traffic encountered

Instruments, apparatus, and equipment assembled or developed for the project

Stop watches
Thermometers
Psychrometer
Potentiometer
Tire pressure gauge
Anemometer
Three battery clocks:
1. For measuring engine hours
2. For measuring actual running time
3. For measuring braking time

Gasoline storage tank and service station pump
Gasoline tank stick gauge graduated in 0.2 gal. units
Gasoline meter recording gasoline in 0.01 gal. units
Oil stick gauge graduated in 0.1 quart units
Steering effort and steering angle integrator
Engine revolution counter and speed indicator
Electric counters for clutch and brake applications
Air speed indicator
Ball bank indicator
Rotary selector switch to speed up reading temperatures of various thermocouples with potentiometer
Fuel and exhaust line pressure gauge
Manifold vacuum gauge
Certain special equipment includes:
Space-time recorder
Stromberg exhaust gas analyzer
Hydraulic brake pressure gauge
Tapley decelerometer
Tire depth gauge
Fifth wheel and Weston tachometer
Integrating dynamometer
Soil penetrometer, impact type and Proctor type
Firestone six-element sensitive recording contact accelerometer to measure road roughness and correlate with road profilometer study

In addition to the above items, equipment is available at the laboratory and at the Highway Commission garage to make many checks and adjustments in tuning up the motors and in keeping the cars and tires in good standard condition.

Routine tests: Routine tests were started October 12, 1938, and the schedule for the fall season involving 7,100 miles for each year was completed December 30. Similarly routine tests were run during each successive three month period comprising the winter, spring, and summer seasons during 1939. On the test runs a speed schedule was followed to provide for the same uniform cruising speed on from three to six runs over the test route. Accurate gasoline and oil readings were taken at the beginning and end of each run on the tank stick gauge and on the oil stick gauge. Readings were taken from the gasoline meter at the beginning and end of the specially marked sections on the course during each run together with the readings of all the other meters, counters, and temperatures for each car. Tires were measured to determine tread wear at the end of each speed series or about every 700 to 1,400 miles. The crankcase oil was also drained at this time, carefully measured by weighing and a sample taken for analysis. On the basis of the results in the used oil tests conducted by the Iowa Highway Commission testing laboratory the schedule for making oil changes was determined. The oil changes have been made on an average of about every 2,000 miles on the gravel and the concrete car not equipped with an oil filter and about every 4,000 miles on the gravel and the concrete car equipped with an oil filter. The cartridges of the oil filters were changed after about 5,000 miles of use. The cartridges are made of cotton fiber materials and after use were ignited to determine the net ash residue which was a measure of the abrasive ma-

terial collected in the filter. The garage mechanic inspected and lubricated each car at the end of each speed series.

At the end of each season's runs the standard performance tests were conducted on the five mile test course near Ames to determine car condition in regard to fuel economy at speeds from 10 to 70 m.p.h. and the accelerating and decelerating characteristics of the cars.

The results of all tests were summarized as soon after the tests were run as practicable and complete summary tables and charts were prepared from the daily records for each speed series and for the runs for each season. Every effort was made to keep a continual check on the performance of the cars and on the accuracy of the results recorded. While the inspection and the lubrication service was better than that of the average car owner, it was not greatly different from that which fleet operators are now providing. On the basis of these measurements and the general plan of operation just described, the cost differentials so obtained are conservative and probably are the minimum values rather than the average values for the given road conditions. At the same time a mass of information on all phases of car operation has been or will be collected which will, it is hoped, indicate how car costs may be further reduced.

RESULTS

Standard Performance Tests

To determine possible differences in the power and the gasoline mileage characteristics of each of the four cars, standard performance tests were run on the five mile test course north of Ames at the end of the break-in period and before and after each set of seasonal runs. These tests consisted of acceleration and deceleration determinations on a level grade and gasoline mileage determinations for speed, from 10 to 70 m.p.h., on the five mile course.

The results of the acceleration and deceleration tests after the break-in period (Fig. 11) showed that the cars were very close together at that time. Car No. 5 gave slightly higher accelerating rates in the various gears than were obtained for the other cars but its gasoline mileage was also slightly lower than that of the

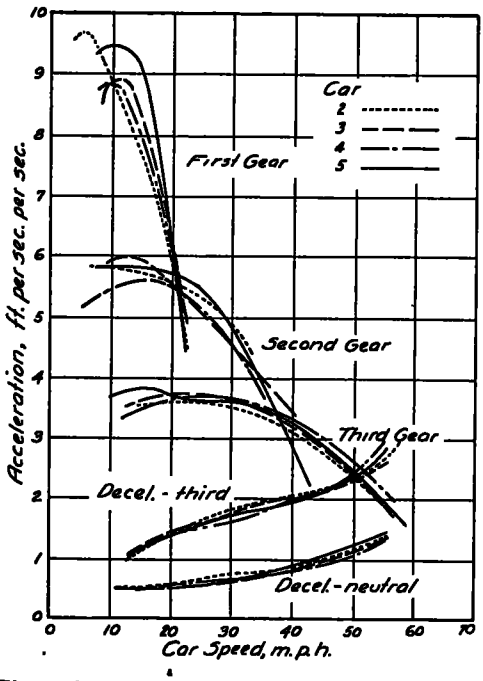


Figure 11. Typical Acceleration and Deceleration curves for four test cars, Fall 1938

other cars (Fig. 12). For Car No. 2 the reverse was true with the gasoline mileage about 1 mile per gallon higher and the accelerating rates slightly lower than for the remaining three cars. While this advantage for Car No. 2 has been evident in practically all the calibration tests during the year, the differences in gasoline mileage have been less than those observed in the routine test runs over the 237 mile gravel course. After a consider-

ation of the results in the standard performance tests and the routine tests, it can be said that the four cars have been kept in good adjustment by the mechanic in the routine and seasonal inspections and have held their adjustments so well during the runs, that the differences in the power and gasoline consumption characteristics for the four cars may be considered to be too small to require that correction factors be applied.

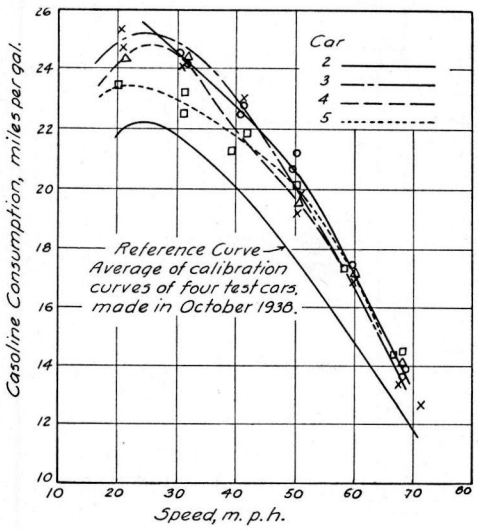


Figure 12. Typical Gasoline Calibration curves for 4 test cars, July 1939, before inspection and servicing as compared to a composite calibration curve, October 1938.

Gasoline Consumption Measurements

The measurement of gasoline consumption on the two surfaces is one of the most critical tests being made. Extra precautions have been taken to assure accurate measurements. The gasoline is measured by three independent methods: (1) the service station pump (Fig. 13) graduated in 1/20 gallon units makes possible measuring the gasoline to an accuracy of about 1/100 gallon, (2) the car gasoline tank stick gauge which can be read to 1/10 gallon, and (3) the fuel meter (Fig. 14) which automatically records the fuel used in 1/100 gallon units. The comparison of the results of the gasoline measurements on the test routes for the spring season, 1939 (Fig. 15), shows that the pump and stick gauge values are practically the same but that the gas meter values averaged about $\frac{3}{4}$ mile per gallon higher on the concrete cars and $1\frac{1}{2}$ miles per gallon higher on the gravel cars than the pump or tank values. An explanation for this difference is that it may have been caused by volatilization and escape from the tank of the lighter fractions of the gasoline as it sloshes around in the tank. Since the

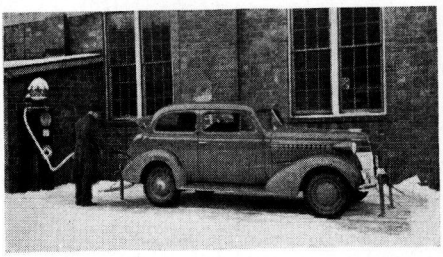


Figure 13. Service Station pump. Also test car on level concrete platform with bumper jacks in position to show method of leveling car for gas tank stick readings.

gravel roads were much rougher than the concrete roads, the losses on the gravel roads should be greater.

The gasoline used in this study was purchased in 400 to 500 gallon lots and samples were tested by the Iowa Highway Commission Testing Laboratory. The gasoline used was a regular grade Standard Oil of Indiana product with an average A.P.I. gravity of 65 in the winter and 59 in the summer, initial boiling point of 84° F. in the winter and 106° F. in the summer, end points of 392° F. winter and 410° F. summer, and an octane rating of 69 to 70 winter and summer.

Since gasoline consumption varies greatly with speed, it is necessary to take

into account speed changes when comparing fuel requirements on gravel and on concrete. For this reason the comparisons of fuel consumption are shown in curve form (Fig. 16) for each car for the first year of routine tests. The average gasoline mileage for the entire year was about two miles per gallon higher on concrete than on gravel. At 25 miles per hour the concrete cars averaged 21½

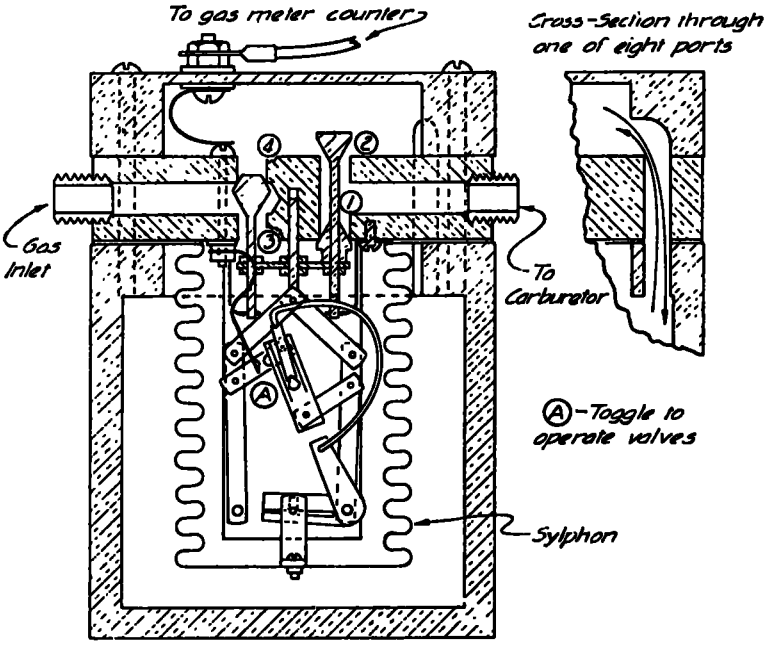


Figure 14. Sectional View of Gas Meter.

Operation of Meter—One Cycle: As Gasoline enters through valve (3), the Sylphon is expanded forcing Gasoline from Chamber surrounding Sylphon through ports, valve (2), and on to Carburetor. When expanded Sylphon trips toggle (A) operating valves, (3) is closed and (1) is opened. Simultaneously (2) is closed and (4) is opened, allowing Gasoline to flow through ports to Chamber surrounding Sylphon. Sylphon is contracted, forcing Gasoline through valve (1) to Carburetor until toggle is again tripped.

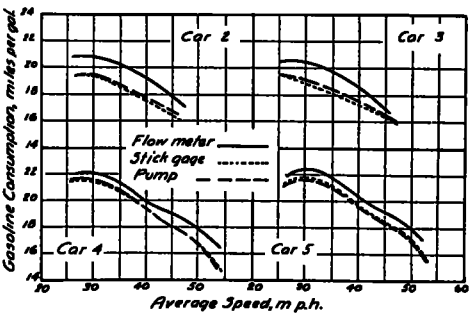


Figure 15. Comparison of Gasoline Consumption measurements on test routes, Spring 1939

miles per gallon and the gravel cars 19½ miles per gallon; at a cruising speed of 50 m.p.h. the concrete cars averaged 18 miles per gallon and the gravel cars 16¼ miles per gallon. At 65 miles per hour, the highest cruising speed maintained on concrete, the average gasoline mileage was 15 miles per gallon and at 55 m.p.h., the highest cruising speed considered safe on gravel, the average mileage was 15¼ miles per gallon. The close check between the values for the two cars

on each route is an indication of the dependability of the cars in giving consistent results and also indicates that the methods used in running the tests are providing the desired accuracy.

The greatest difference between the two surfaces was obtained during or after rains which caused the gravel to

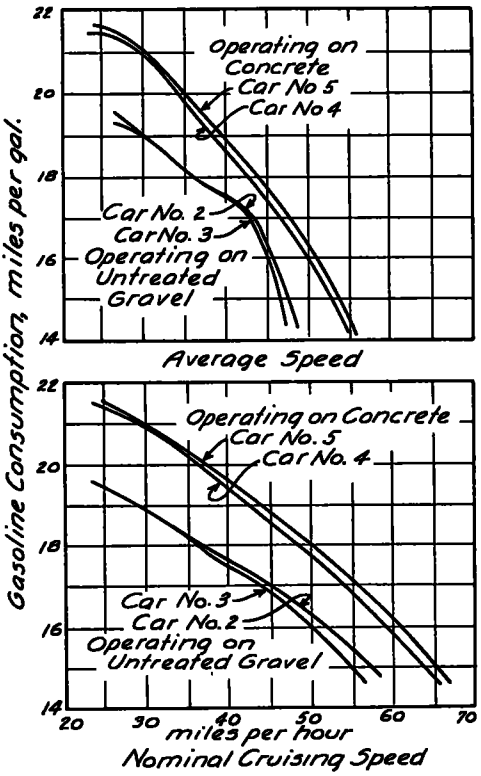


Figure 16. Gasoline Consumption on Concrete and Gravel Surfaces, based on one year of Test Driving, or about 32,000 miles per car.

be soft but which had no effect on the gasoline mileage results on concrete. For the wet gravel an average difference of 3.0 miles per gallon was obtained when compared with dry concrete whereas for the dry smooth gravel the difference was only about 1.0 mile per gallon. On wet concrete the gasoline mileage averaged about 0.5 mile per gallon higher than on dry concrete for speeds of 45 and 55 m.p.h. and was the same wet or dry at 65 m.p.h. Decreased friction on the wet concrete and increased engine efficiency due to higher humidity are factors which may have contributed to the higher values on the wet concrete.

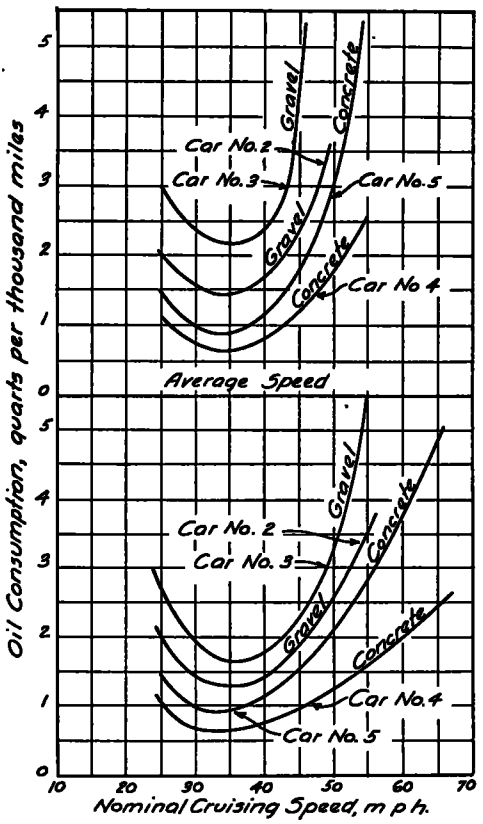


Figure 17. Oil Consumption on concrete and gravel surfaces, based on one year of test driving, about 32,000 miles per car. Cars 2 and 4, oil filter and oil bath air cleaner. Cars 3 and 5, no filter or cleaner.

Oil Consumption

The results for oil consumption (Fig. 17) indicated that the cars with standard equipment used 1¼ to 3 quarts more oil per 1,000 miles on gravel than on concrete at corresponding speeds while for the cars with oil filters this difference

amounted to ¾ to 1½ quarts. The gravel cars used about twice as much oil as the concrete cars in terms of oil added but not including oil replaced when changing oil. The value of oil filters in reducing oil consumption, in keeping down cylinder wear, bearing wear, and ring sticking was clearly demonstrated. The oil consumption in terms of oil added for the cars with oil filters was from 25 to 50 per cent lower than for the cars without oil filters.

The oil used in this investigation was a Standard Oil of Indiana product, Polarine S.A.E. 20 during the fall, winter, and spring and S.A.E. 30 during the summer season. The new S.A.E. 20 oil had a Sayboldt viscosity of 50 at 210° F., flash point 425° F., fire point 485° F., pour test −5° F., color N.P.A. 3, A.P.I. gravity at 60°/60° F. of 25.8, carbon residue 0.08 per cent, and a neutralization number of 0.01 according to samples tested by the Iowa Highway Commission Laboratory.

The oil consumption results are consistent in one respect, which is that the oil exposed to the most dust or the "dirty" oil was used faster than clean oil. The relative cleanness of the oil was verified by the analysis of the used oil (Table 3). The oil in Gravel Car No. 3, not equipped with an oil filter, was clearly shown by the laboratory tests to be in the worst shape of the oil in any of the cars. It was higher in carbon, abrasives, and asphaltenes (insoluble in naphtha) than any of the oil in the other cars although it was changed every 2,000 miles instead of every 4,000 or 5,000 miles as has been the practice in changing oil in the cars with oil filters. During the first three seasons covering about 25,000 miles, the oil filter on the Gravel Car No. 2 removed 296 grams of abrasive from the oil and the filter on the Concrete Car No. 4 removed 135 grams. But what is more important the asphaltenes or gummy residues in the oil were held to the low value of 0.22 per cent after 4,404 oil miles on Gravel Car No. 2 as compared to 1.02 per cent on Gravel Car No. 3 after 2,428 oil miles.

The importance of "clean" oil as it relates to oil consumption and good engine performance was demonstrated by Car No. 3. As stated before, the oil in Car No. 3 was shown by test to contain more carbon, abrasives, and asphaltenes than the oil in any of the other cars. Also, Car No. 3 used more oil than any of the other cars but it was not considered to be excessive at the time of the April inspection. Shortly thereafter (car mileage 17,800 miles) Car No. 3 suddenly started to use oil so fast that the oil fumes attracted the attention of the driver. Careful examination revealed no leaks but heavy oil streaks on the underparts of the car indicated that the oil was forced out of the crankcase through the breather. Also a sample of the oil taken from the crankcase was saturated with carbon. It seemed evident that some of the oil rings were clogged and stuck, thereby permitting the blowby past the piston during the power stroke to force the oil out of the crankcase through the breather. The pan was therefore dropped and the head removed. The pistons and head for cylinders 1, 2, and 5 were found to be oily and more than half of the holes in the oil rings for these cylinders were clogged, thus indicating that the rings in these cylinders were stuck and were the cause of the trouble.

The cylinders in Car No. 3 were measured while the head was off and the maximum wear at top of the ring stroke was found to be 0.003 in. (change in diameter). This tapered down to less than 0.001 in. at a point 1 in. below top of ring stroke. In other words the wear on the cylinders in Car No. 3, which were exposed to more dust than the cylinders of any other car, may be considered to be

TABLE 3

SAMPLE RECORD OF USED OIL ANALYSIS FOR OIL TAKEN FROM CRANKCASE, AIR CLEANER AND OIL FILTER DURING SPRING SEASON, 1939

Car No. and surface	Car mileage	Nominal speed	Oil miles	Saybolt viscosity seconds, 210°F.	Dilution	Water	Insoluble in benzol	Carbon	Abrasive	Insoluble in 86° benaphtha	Acid No. mg./KOH/gm. oil	Material analyzed	Remarks
					Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*			
	21,810	..	3,650	..	...	...	0.110[1]	.....	.....	.....	....	Air cleaner res.	Car equipped with oil filter and oil bath air cleaner.
	22,549	25	739	51	1.6	Nil	0.013	0 013	0.000	0.076	0.13	S.A.E. 20	
	23,749	45	1,939	52	1.2	Nil	0.045	0 044	0.001	0.225	0.16	S.A.E. 20	
No. 2,	25,250	35	3,440	52	1.2	Nil	0.016	0.016	0.000	0 147	0.14	S.A.E. 20	
Gravel...	26,214	55	4,404	53	0.8	Nil	0.002	0.000	0.002	0.219	0.15	S.A.E. 20	
	26,547	..	8,387	..	...	...	0.040[1]	0.006[1]	0.034[1]	.....	....	Crankcase res.	
	26,547	..	8,387	..	...	...	.....	.....	175 gr.	.....	....	Filter pack ash	
	26,547	..	25,540	..	...	...	.....	.....	296 gr.	.....	....	Total ash[2]	
	23,100	..	1,908	..	...	...	0.199[1]	.....	.....	.....	....	Air cleaner res.	Car equipped with stack air filter and no oil filter.
No. 3,	24,568	35	1,468	50	1.2	Nil	0.039	0.039	0.000	0.626	0.12	S.A.E. 20	
Gravel...	25,528	55	2,428	51	0.8	Nil	0.121	0.057	0.064	1.017	0.16	S.A.E. 20	
	25,813	..	8,309	..	...	...	0.197[1]	0 027[1]	0.170[1]	.....	....	Crankcase res.	
	22,465	..	5,323	..	...	...	0.003[1]	.....	.....	.....	....	Air cleaner res.	Car equipped with oil filter and oil bath air cleaner.
	23,645	55	1,180	53	1.2	Nil	0.026	0.016	0.010	0.293	0.16	S.A.E. 20	
	25,057	45	2,592	53	1.0	Nil	0.012	0.007	0.005	0.358	0.20	S.A.E. 20	
No. 4,	25,999	55	3,534	53	1.0	Nil	0.011	0.005	0.006	0.411	0.17	S.A.E. 20	
Concrete	26,239	..	8,887	..	...	...	0.021[1]	0.012[1]	0.009[1]	.....	....	Crankcase res.	
	26,239	..	8,887	..	...	...	.....	.....	77 gr.	.....	....	Filter pack ash	
	26,239	..	25,080	..	...	...	.....	.....	135 gr.	.....	....	Total ash[2]	
	22,529	..	2,603	..	...	...	0.016[1]	.....	.....	.....	....	Air cleaner res.	Car equipped with stack air filter and no oil filter.
No. 5,	23,702	55	1,173	51	1.2	Nil	0.030	0.015	0.015	0.644	0.20	S.A.E. 20	
Concrete	25,132	45	2,603	52	1.6	Nil	0.035	0.023	0.012	0.910	0.20	S.A.E. 20	
	26,306	..	8,916	..	...	...	0.147[1]	0.058[1]	0.089[1]	.....	....	Crankcase res.	

[1] Percentages based on 4.5 quarts oil in crankcase.

[2] Total ash from four used filter packs.

so small that the dust itself was not a direct cause of the ring sticking. It appears more likely that the dust acted as a catalyst and hastened the formation of carbon and sludge or gummy residues which seemed to be the direct cause of the clogging of the rings in Car No. 3.

Tire Wear

All measurements of tire wear were made on the basis of tread depth using a depth gauge graduated in 0.001 in. units (Fig. 18). The results for tire wear are given in curve form (Fig. 19). As in the case of gasoline and oil consumption measurements, the concrete surface again indicated its superiority. At a speed of



Figure 18. Method of measuring tire wear; depth gauge measures tread depth

25 m.p.h. the average rate of wear on gravel was practically double that on concrete, at 50 m.p.h. this difference was cut in half and there is an indication that at 65 m.p.h. the rates for the two surfaces would have been the same. These differences in the rate of wear at the given speeds can be attributed to the type of road material. On gravel surfaces the tires "plow" through the loose gravel and roll over the loose particles with "ball-bearing" type action. The contact of the tire with the gravel surface depends on the condition of the surface and the speed of the car. It is not surprising in view of the basic differences in the surfaces that the amount of wear and the shape of the wear curve is different on gravel than on the hard smooth concrete.

While the increased tread wear on the gravel as compared to the concrete surfaces is an important element in the differences in tire costs on these surfaces, items of equal importance not only in regard to cost but especially in regard to the annoyance or inconvenience which they cause are the many punctures, blowouts and other tire failures on the gravel surfaces. The record of the first set of tires on the gravel cars (Table 4) shows that there were 51 punctures in a total of 63,820 miles of driving whereas

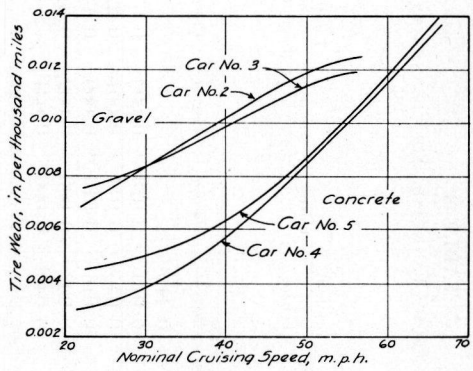


Figure 19. Comparison of tire wear on concrete and untreated gravel, representing one year of test driving or about 32,000 miles per car.

there was only one puncture in a total of 72,290 miles of driving of the concrete cars which on November 1, 1939, were still equipped with the first set of tires. Only two out of the ten tires on the gravel cars failed by tread wear and eight out of ten failed by blowouts, breaks in the fabric on the inside of the tire caused by punctures and stone bruises, and by separation of the plies along the shoulder caused by stone bruises and the severe punishment given the tires in 25,000 miles or more of travel on rough, washboarded gravel surfaces.

This experience with punctures and

TABLE 4

CONDITION OF TIRES WHEN RETIRED FROM SERVICE ON GRAVEL CARS, AUGUST 31, 1939

Car No.	Car mileage	Tire No.	Tire mileage*		Av. tread depth, in.		Number of punctures	Reason for retirement
			Concrete	Gravel	When new	When retired		
2	31,973 3	398D203	5,396.2	20,711.7	0.324+	0 054	5	Tire at point of blowing out.
2	32,223.9	399D702	3,733 3	17,563.7	0.323+	0 098	4	Tire at point of blowing out.
2	32,223.9	399D603	4,088 3	24,056.2	0.324	0:047	3	Difficulty in reading tread depth.
2	31,307.9	395D303	4,522.2	21,853.9	0.322	0.053	4	Tire blew out.
2	31,973.3	394D803	5,295.6	21,173.4	0.328+	0.055	5	Tire blew out.
3	31,746 8	398D603	4,500.2	22,812 5	0.326	0 046+	5	Difficulty in reading tread depth.
3	31,746.8	396D703	4,549.6	21,551 8	0.324+	0.043+	9	Unsafe for further use.
3	31,673.4	396D303	2,436.2	19,139.4	0.327	0.105	3	Tire at point of blowing out.
3	30,826.4	394D703	3,877.8	22,813.3	0.324	0 058	5	Tire blew out.
3	31,746.8	393D403	3,940.5	21,262 9	0.324	0 052	8	Unsafe for further use.

Car No.	Car mileage	Tire No.	Boots, No.	Cross patches, patches, No.	Puncture holes, No.	Holes raveling, No.	Fabric cuts, No. and size	Vulcanized Patches, No.	Condition of tread face
2	31,973.3	398D203	2	1	5	..	1—½″	..	Some checking.
2	32,223.9	399D703	..	1	4	1	1—1″	1	Some checking.
2	32,223.9	399D603	..	..	3	..	2—¼″, ½″	..	Some checking.
2	31,307.9	395D303	1	1	4	..	1—4″	..	Badly checked.
2	31,973.3	394D803	1	1	5	..	1—3″	1	Good.
3	31,746.8	398D603	..	1	5	..	1—½″	..	Badly checked.
3	31,746.8	396D703	..	1	9	1	1—¼″	..	Badly checked.
3	31,673.4	396D303	..	1	3	1	1—½″	1	Fair.
3	30,826.4	394D703	1	..	5	..	2—8″, 1″	..	Good.
3	31,746.8	393D403	..	..	8	3	1—¼″	..	Badly checked.

* The total tire mileage is the sum of the mileage on concrete and on gravel.

NOTE: Condition of tires on concrete cars on Oct. 1, 1939.

Average mileage per tire = 28,915.

Punctures—1.

Tire failure—1 (approximately 6 inches of tread separated from tire exposing fabric.)

Average tread depth when new = 0.324+.

Average tread depth Oct. 1, 1939 = 0.075.

tire failures is not new. It is a characteristic of the road which cannot be corrected except by providing a smooth firm surface. The rough loose gravel jars nails, etc., from the wagons, cars, and trucks and the loose gravel holds the nails. On hard smooth surfaces the few nails which are dropped are easily swept off the surface by fast moving traffic. In Figure 20 the assortment of nails, spikes, and staples picked up by the gravel cars are shown. Since tire pressures were checked every 60 miles, many punctures were detected before the tire went flat. In no case were the casings rim cut by running on a flat tire without the driver knowing it. The high mileage

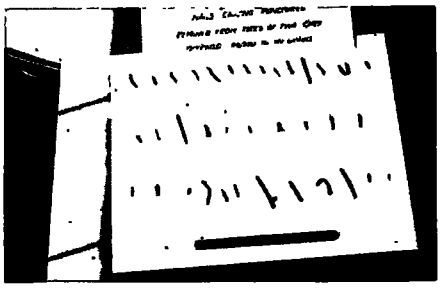


Figure 20. Nails causing punctures removed from tires of two cars operated 55,000 miles on untreated gravel.

of the gravel tires and of the concrete tires may be attributed to care in driving, to rotation of the tires from one wheel position to the next around the car, thus keeping wear equalized, and to keeping the tires in balance and at standard air pressure.

Engine Wear, Brake Wear, and Maintenance Items

The repairs and replacements on the four test cars during and at the end of the first year of test driving are given in Table 5. At that time, measurements were taken to determine the wear in the cylinders, pistons, connecting rod bearings, main bearings and crankshafts (Table 6). At the end of each season (approximately every 9,000 miles) the wheels were removed for brake inspection which provided an opportunity to determine the wear of the brake linings and brake drums (Table 7).

The cars were given a large amount of preventive maintenance. Approximately every 1,000 miles they were washed if caked over with mud, especially on the underparts of the car, or if excessively covered with dust and dirt. They were then lubricated, serviced, and the performance of the motor and brakes checked by the mechanic. At the end of each season, the pan was dropped, the wheels were removed and a more complete inspection was made. At the end of the first year's driving (36,000 miles), the entire car was checked over carefully and parts cleaned and replaced if necessary as shown in Table 6. Since the oil consumption rate in Cars 3 and 5 had reached a rate close to 4 quarts per 1,000 miles by the end of the year and since there was evidence of clogged rings, it seemed advisable to change rings again in Car No. 3 and also to change rings in Car No. 5. No changes were made in the rings for Cars No. 2 and No. 4 because the oil consumption for these cars was less than 2 quarts per 1,000 miles and the motors were clean.

The brake linings on both gravel cars were badly worn and needed replacement. The rear brake drums of the gravel cars were scored due to the pressure of sand and grit and were replaced because it was cheaper and better to install new drums than to machine the old ones to a smooth and round condition.

The advantage of oil filters in keeping down engine wear is shown in Table 5. The cylinders and bearings of Cars No. 2 and No. 4 which were equipped with oil filters were in excellent condition after 36,000 miles of travel whereas Cars 3 and 5 which were not equipped with oil

TABLE 5

REPAIRS AND REPLACEMENTS ON FOUR TEST CARS DURING AND AT END OF FIRST YEAR OF TEST DRIVING
(Car Mileage per Car—36,200±)

Car No.	Operated on	Repairs, replacements, and car mileage
Common to all four cars		New battery and cables (35,000± mi.). New transmission and differential grease (four changes). New spark plugs (17,200 mi., and 36,200 mi.) New ignition wiring (36,200 mi.). New distributor points (17,200 mi. and 36,200 mi.). New fuel pump diaphragm and valves (36,200 mi.). New generator brushes (36,200 mi.). New gaskets (replacement of gaskets at each seasonal inspection).
2	Gravel	New rear brake drums (36,300 mi.). New brake linings (all four wheels at 36,300 mi.). New battery cables (12,600 mi.). Starter switch, weld exhaust pipe (36,300 mi.).
3	Gravel	New rear brake drums (35,800 mi.). New brake linings (all four wheels at 35,800 mi.). New piston rings (17,800 mi. and 35,800 mi.). New exhaust valve and tappet (35,800 mi.). New spindle bushings (26,000 mi.). New front wheel bearing (33,000 mi.). New headlight lens (34,300 mi.). Repair battery basket (35,800 mi.).
4	Concrete	No replacements except as noted above.
5	Concrete	New headlight lens (19,900 mi.). New piston rings (36,100 mi.).

TABLE 6

BEARING AND CYLINDER WEAR FOR ENGINE OF EACH CAR AT END OF ONE YEAR OF TEST DRIVING
(Wear in Thousandths of an Inch)

Car No.	Car mileage	Cylinders, change in diameter		Pistons	Connecting rod bearings[1]	Main bearings[1]	Crankshaft on throws, change in diameter	Car driven on
		At top of ring-stroke	1-inch below top of ring-stroke					
2[3]	36,300	1.0	0.5	0.0	6.0	....[2]	1 0	Gravel
3	35,800	4.5	2.5	0.0	12.5	14.6	3.1	Gravel
4[3]	36,100	1.5	0.5	0.0	0.0	0.0	0.5	Concrete
5	36,100	1.8	1.5	0 0	3.0	2.5	1.5	Concrete

[1] Measured thickness of shims removed.
[2] Not removed, bearing tight, no appreciable wear observed.
[3] Car equipped with oil filter and oil bath air cleaner.

filters indicated a substantial increase in wear. This was especially true of Gravel Car No. 3 which has shown the greatest effect of dust on oil consumption and also on wear.

Summary of Operating Costs for the Two Gravel Cars and the Two Concrete Cars

The operating costs during the first year of operation on the test routes have been summarized on the basis of the cost items which were measured during or at the end of each speed series (Table 8) and on the basis of materials used, parts replaced, and labor costs charged against each car during or at the end of the year (Table 9). No attempt was made to evaluate wear for parts not replaced or for depreciation of the cars except in the case of the tire costs. The tire costs were based on tread depth wear, actual repair costs for tires and tubes, a 10 per cent salvage value for casings with the tread worn smooth but with no breaks in the fabric, and a 50 per cent salvage value for tubes used in the original set of tires and now used in the second set of tires.

The local retail prices were used for all items. The prices of the more important items were as follows: gasoline, 18.5 cents per gallon, oil 25 cents per quart, tires $15.70 each, tubes $3.65 each, repair of punctures 50 cents each, and labor charges for inspection, servicing, and all repairs $1.00 per hour.

The cost of repairs does not include that due to accidents in which the cars were involved. During the first year, in which the test cars were driven more than 150,000 miles, they were involved in four minor accidents, all of which

TABLE 7

BRAKE SHOE AND DRUM WEAR OF ROAD TEST CARS

Car mileage ±	Position	Accumulated average wear, 0.001 in.											
		Car 2 (Gravel)			Car 3 (Gravel)			Car 4 (Concrete)			Car 5 (Concrete)		
		Front shoe	Rear shoe	Drum	Front shoe	Rear shoe	Drum	Front shoe	Rear shoe	Drum	Front shoe	Rear shoe	Drum
10,600	LF	12	2	0	12	4	0	11	2	0	8	1	0
17,500		23	2	1	26	6	2	20	4	0	16	1	0
26,200		28	4	2	37	9	3	26	4	1	20	1	0
36,200		37	5	4	47	11	3	33	8	2	25	2	2
10,600	RF	16	4	0	7	0	0	0	1	0	0	0	0
17,500		27	8	2	17	2	3	15	1	0	15	3	0
26,200		35	8	3	25	3	3	21	2	1	20	3	0
36,200		47	15	4	33	12	3	27	5	2	23	3	0
10,600	RR	15	2	1	23	7	3	4	0	0	6	0	0
17,500		29	6	3	35	9	5	14	0	0	12	1	0
26,200		45	11	6	53	16	12	17	2	0	18	1	0
36,200		64	13	8	68	25	26	17	3	2	21	2	0
10,600	LR	14	3	2	16	2	4	9	3	0	6	2	0
17,500		35	7	6	27	6	4	10	3	1	10	3	0
26,200		53	14	11	42	10	8	16	3	1	13	4	0
36,200		70	21	16	66	17	10	20	13	2	18	4	0

All initial readings taken at a car mileage of 1000± miles.

NOTE: Drum wear readings are changes in inside diameter of drum.

occurred on the gravel route with the gravel cars. Two of them were very minor, the repair costs amounting to $2.50 in one and $4.50 in the other. The other two were collisions on the left front and side and involved more extensive repairs totaling $71.32 for the accident to Car No. 2 and $46.50 for the accident to Car No. 3. Bent and broken parts were in most cases replaced with new parts and every effort was made to restore the car to as good a condition in the parts affected as possible. Several checks of the wheel and frame alignment were made later to assure that no changes in alignment had developed which had not been corrected. No repairs have been reported since the accidents occurred which might be considered as due to the accidents.

TABLE 8

GASOLINE, OIL, AND TIRE COSTS AT VARIOUS CRUISING SPEEDS FOR ONE YEAR OF DRIVING ON THE GRAVEL AND CONCRETE TEST ROUTES

Surface	Cruising speed	Operating cost, cents per mile			
		Gas	Oil	Tires, from tread wear	Total gas, oil, and tire cost
Portland cement concrete..	25	0 860	0.030	0 080	0.970
	35	0.912	0.020	0.104	1.036
	45	0 989	0.031	0 149	1 169
	55	1.088	0 055	0.208	1.351
	65	1.242	0 090	0.275	1.607
Untreated gravel.....	25	0.954	0.059	0.178	1 191
	35	1.014	0.036	0.214	1 264
	45	1.088	0.053	0.248	1.389
	55	1.208	0.112	0.284	1.604

TABLE 9

OPERATING COSTS OF ALL ITEMS MEASURED FOR ONE YEAR OF DRIVING ON THE GRAVEL AND CONCRETE TEST ROUTES

Car No. and surface	Average speed	Operating cost, cents per mile										Total cents per mile
		Gas	Oil	Tire cost		Maintenance						
				Wear	Repairs	Lubrication	Washing	Routine inspection		Seasonal or major repairs		
								Labor	Parts	Labor	Parts	
No. 2, gravel..	35.8	1 026	0.036	0.300	0.046	0.125	0.073	0.120	0 042	0.168	0.138	2.074
No. 3, gravel..	35.9	1.033	0.054	0 297	0.054	0 129	0.072	0 116	0.035	0.312	0.184	2.286
No. 4, concrete	40.7	1.000	0.021	0.174	0.000	0.107	0.038	0 109	0.040	0.094	0.082	1.665
No. 5, concrete	40 7	0.985	0.030	0.176	0.003	0.116	0.038	0 109	0.025	0.179	0.101	1.762

The three important cost items affected by variations in speed for which definite measurements were made at each speed, were the cost of gasoline, oil and

tire wear. For these items the costs at corresponding speeds averaged ¼ cent per mile higher on gravel than on concrete. When tire repairs and all the maintenance items for which costs were kept, such as lubrication, washing, and repairs, were included, the costs averaged ½ cent per mile higher on gravel than on concrete. In the latter case the average speed for the gravel cars was 35.8 miles per hour as compared to 40.7 miles per hour for the concrete cars. Hence, the difference for equal speeds for all of the above items would be greater than ½ cent per mile.

The general inspection and repairs made at the end of the first year's operation were intended to bring the cars to as nearly the same condition as practicable. However, as stated before there were certain maintenance and depreciation items which were not evaluated but which the measurements of wear and the appearance of the worn parts definitely indicated were higher for the gravel cars than for the concrete cars. The effects of road roughness, dust, loose gravel, and the extra wear and tear caused by these items, should show up at a later date in extra repair and maintenance costs on the gravel cars.

The operating costs as related to speed are not so complete as they should be to make true comparisons because it is not feasible to evaluate variations in maintenance costs due to speed in field tests of this type. The values for the cost of gasoline and oil used and for the cost of tires as they relate to speed (Table 8) are accurate and are of special interest when determining the extra costs due to speed. On the concrete route the cost of gasoline, oil, and tires amounted to 0.970 cent per mile at 25 m.p.h. and 1.607 cents per mile at 65 m.p.h., an increase of 65 per cent. On the gravel route these costs amounted to 1.191 cents per mile at 25 m.p.h. and 1.604 cents per mile at 55 m.p.h., an increase of 35 per cent as compared to a 40 per cent increase for similar speeds on concrete. For all the cost items as reported in Table 9, the difference in costs due to using an oil filter amounted to 1/10 cent per mile on concrete and 1/5 cent per mile on gravel for this particular type of operation and with the care given the filter as recommended by the manufacturer.

While these summaries of operating cost give some indication of the probable cost differentials which will finally be obtained for the two surfaces, the total mileage is still too low (36,200 miles per car) to reflect the full difference.

Summary of Measurements related to Speed, Time, Temperature, and Pressures

The summary of the various readings taken from the instruments on the panel board for each car are given in table form for the 1939 winter series (Table 10). This information may be used for many different purposes; for example, it is interesting to note that for a cruising speed of 25 miles per hour, the actual average speed including travel through towns is about the same as the cruising speed. But when a cruising speed of 65 m.p.h. is attempted, the average speed is reduced to 54 m.p.h., partly due to the effect of traffic interference and partly to the slower speeds required in towns and cities.

The temperatures of various parts of the car under continuous running are also of interest. The crankcase oil temperatures indicate the effect increased speed has on oil temperatures and provides information of value to investigators studying bearing failures and cylinder wear. The cooling water temperature seemed to hold fairly uniform as the speed was increased. The exhaust manifold temperatures running well above 1,000° F. for the higher speeds even dur-

TABLE 10

SUMMARY OF VARIOUS MEASUREMENTS RELATED TO SPEED, TIME, TEMPERATURE, AND GAUGE PRESSURES
(Winter Runs—1939)

Car No.	Nominal speed	Miles	Motor time	Run time	Average speed	Clutch application	Brake application	Braking time	Thermocouples										Pressures		
									1	2	3	4	5	6	7	8	9	10	Exhaust	Fuel	Man.
2	25	947.2	33:16	33:00	26.8	474	309	0:21	169	155	134	160	803	23	95	...	...	95	0.2	2.4	15.9
3	25	946.8	33:32	33:04	26 4	514	399	0:16	150	149	128	153	918	23	76	98	...	95	0.1	3.0	17.4
4	25	468.1	18:14	17:19	27.0	121	65	0:04	138	129	118	163	1,001	36	70	...	...	72	0.2	2.6	17.4
5	25	468.1	18:11	17:19	27.0	200	131	0:05	136	113	102	152	917	36	63	92	...	70	0.2	2.8	17.3
2	35	2,608.9	76:05	74:56	34.8	.....	1,162	1:18	173	150	129	131	1,127	22	69	...	...	68	0.3	2.8	15.0
3	35	2,612.3	76:30	75:10	34.7	1,428	1,255	1:25	167	149	132	131	998	22	73	98	...	68	0.3	3 1	15.5
4	35	1,167.5	34:40	34:30	33.8	346	323	0:11	163	136	124	137	675	48	86	...	...	83	0.3	2.8	17.4
5	35	1,167.5	34:55	34:40	33.6	347	255	0:10	163	132	122	142	579	48	84	108	110	79	0.2	2.8	16.5
2	40	1,894.0	51:28	50:04	37.9	.....	1,134	1:19	177	152	136	134	998	23	66	...	...	65	0.4	3.0	14.0
3	40	1,896.2	51:32	50:08	37.8	933	1,197	1:12	178	153	138	130	1,037	23	76	100	...	69	0.4	3.2	14.2
4	45	2,566.1	64:18	62:39	41.0	942	778	0:41	177	137	127	122	818	22	72	...	...	66	0.5	2.6	15.0
5	45	2,562.5	64:17	62:56	40.8	849	715	0:22	176	137	128	131	912	22	74	103	102	69	0.5	2.7	14.2
2	50	710.7	19:15	18:57	37.5	556	546	0:31	182	150	131	122	1,319	34	73	...	...	67	0 7	2.5	10.6
3	50	712.8	19:33	18:36	38.3	664	564	0:31	180	151	133	128	1,136	34	82	102	...	73	0.7	3.2	11.4
4	55	1,627.0	36:45	35:03	46.5	621	556	0:33	194	147	137	122	900	25	79	...	...	70	0.8	2.6	13.2
5	55	1,629.6	36:29	35:04	46.4	624	483	0:23	194	145	136	134	997	25	80	112	114	71	0.8	2.8	12 0
4	65	694 5	13:39	13:09	52.9	316	299	0:14	210	145	134	106	902	38	79	...	...	71	1.5	2.4	9 2
5	65	694.5	13:42	13:15	52.7	282	298	0:11	208	146	137	116	795	38	80	124	131	77	1.5	2.7	9.2

All time recorded in hrs. and min., Example 35:16.
Thermocouple temperatures are in degrees Fahrenheit.

1. Crankcase.
2. Radiator inlet.
3. Radiator outlet.
4. Intake manifold.
5. Exhaust manifold.
6. Air temperature.
7. Carburetor intake.
8. Transmission.
9. Differential.
10. Gasoline meter.

ing the winter are of interest. Also, the temperatures of 131° F. for the differential and 124° F. for the transmission with an air temperature of 38° F. are noteworthy. It is significant that the temperatures on the gravel cars for many of the items are higher than the temperatures for the concrete cars. The exhaust back pressure increased slightly with speed. The fuel line pressure remained the same and the manifold vacuum decreased with speed. At the same speed the manifold vacuum was lower on the gravel cars than on the concrete cars because more fuel was required on the gravel than on the concrete.

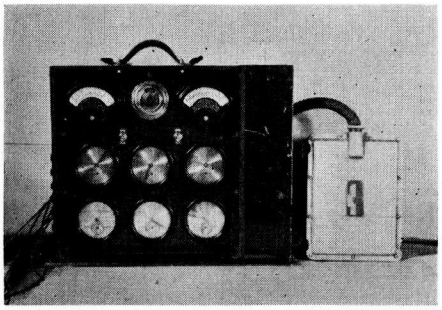


Figure 21. Close-up view of Firestone six-element sensitive recording contact accelerometer.

OPERATING CHARACTERISTICS AND ROAD CHARACTERISTICS RELATED TO SAFETY, COMFORT, AND EASE OF DRIVING

Measurements of Road Roughness

An important characteristic of a road as it relates to motor vehicle operation is the relative roughness of the road surface. This roughness determines the riding quality of the road which is the basis on which the public judges the road's quality. In extreme cases it may be related to safety and ease of driving. At the same time extreme roughness is responsible for a variety of forces, impacts and vibrations which increase operating costs in extra fuel and oil costs, and extra maintenance expense. Gravel roads are frequently rough and especially in dry weather develop long stretches of rhythmic corrugations which are certain to affect the lives of certain parts of the engine and of the car. In this investigation two methods have been used to measure road roughness, one by means of the Firestone riding comfort meter which is a sensitive recording type of multiple-element contact accelerometer (Fig. 21), and the other by means of the road profilometer (Fig. 23) especially

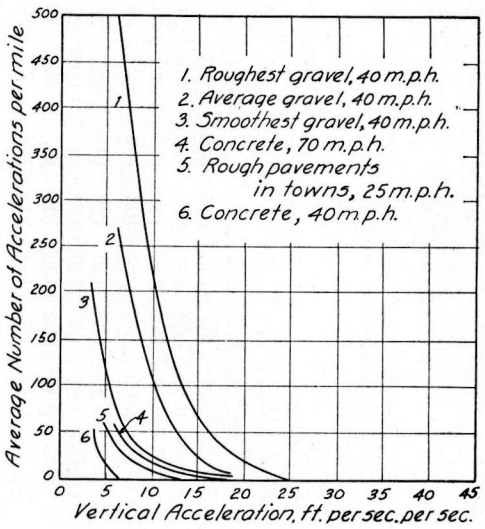
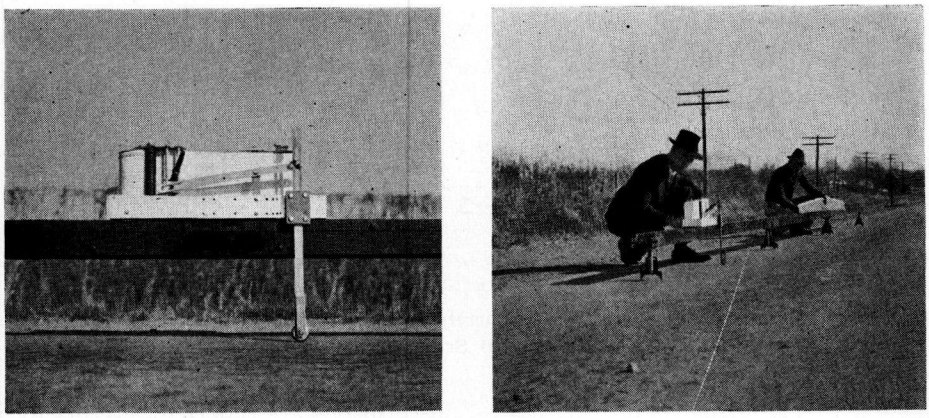


Figure 22. Typical Results with Riding Comfort Indicator

designed for this project. The Firestone riding comfort meter as used in these tests records road roughness in terms of total vertical accelerations varying from 5 ft. per sec. per sec. to 55 ft. per sec. per sec. The results of tests on both gravel and concrete and rough city pavements are given in curve form (Fig. 22). An interesting finding, as indicated by these curves, was that the smoothest gravel road at 40 m.p.h. was considerably rougher than the average concrete pavements at 70 m.p.h. These curves show quite clearly how rough a corrugated

gravel road can be as indicated by test results in which 500 accelerations of 6 ft. per sec. per sec. were recorded per mile, and 50 accelerations of 16 ft. per sec. per sec. per mile at a speed of 40 m.p.h. On concrete the accelerations rarely exceeded 5 ft. per sec. per sec. at 40 m.p.h. Tests by Professor Jacklyn at Purdue University have shown that accelerations greater than 10 ft. per sec. per sec. are definitely uncomfortable. In one series of tests conducted several years ago over



A B

Figure 23. Pictures of profilometer track, ten-foot straightedge, and recording device being operated on a concrete road

undulating grades, the writer found that an acceleration of 5 ft. per sec. per sec. was large enough to cause one of the passengers to be car sick. On three or four occasions during the year when the gravel roads were badly washboarded, several of the driver-observers were reported as car sick as a result of traveling over several hundred miles of rough gravel. For vertical accelerations greater than the force of gravity and with such values the tires are not in contact with the road. It was easy to observe considerable bouncing of the tires from crest to crest of the corrugations on a rough gravel road. How seriously this may effect the life of the gravel cars remains to be seen.

To make comparisons of the variations in roughness of the two road surfaces, accurate profiles were made of 100-ft. sections of road using the road profilometer which provided records as shown in Figure 24. From the offsets for various chord lengths over the 100-ft. test sections sampled, vertical accelerations were computed for an assumed speed of 40 miles per hour and on the basis of the further assumption that neither the surface nor the wheel or body rolling over it were deformed during the operation. The computed vertical accelerations (Fig. 25) obtained in this way were 2 to 4 times greater than those obtained with the Firestone riding comfort meter which was attached to the floorboards in the center of the car. These large differences in vertical accelerations are reasonable and indicate the extent to which tires, springs, and shock absorbers remove a large part of the road roughness by the time these forces reach the chassis of the car.

During the second year of operation it is proposed that additional studies be

made with the riding comfort meter and the road profilometer on both test routes and that these results be correlated with measurements by the Public Roads Administration roughometer which is being developed for the purpose of obtaining a standard measure of road roughness for the entire length of road under investigation.

Driver-Observer Observations of Road Characteristics Related to Cost, Road Maintenance, and Safety

In a careful study of this type involving 150,000 miles of driving over the same test routes in all kinds of weather, it is to be expected that many interesting observations related to cost, road maintenance, and safety were made. These

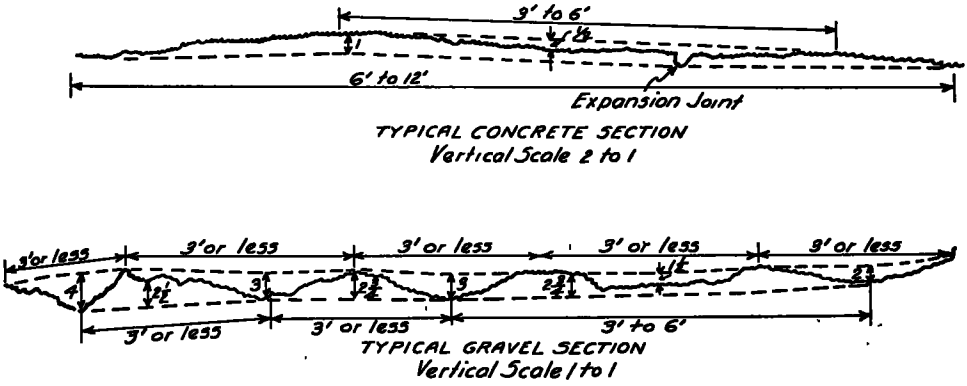


Figure 24. Profilometer measurements on Concrete and Gravel test Routes. Horizontal Scale: 1 in. = 1.07 ft., Vertical Scale in 1/10 in. units

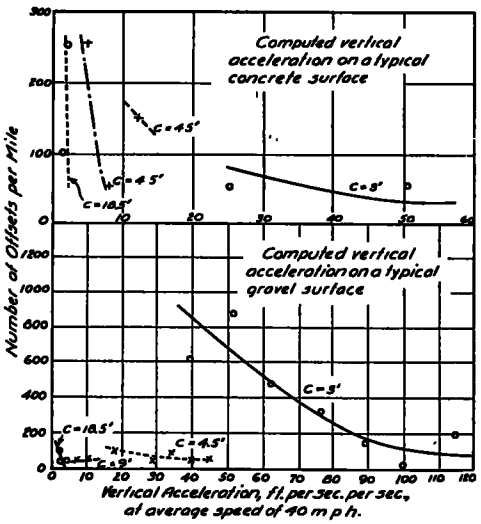


Figure 25. Road Roughness Study, Computed vertical accelerations based on offset measurements for various chord distances.

observations are reported here briefly as a matter of record only, with little or no comment. In many cases they are statements of the intangibles in road comparisons which the highway engineer must weigh in the light of his own judgment and experience:

1. With low crown and heavy rains, pools of water formed on the gravel roads. Large amounts of water and gravel were splashed into the motor compartment. In several cases cars were stalled under these conditions and motors were found to be very wet and dirty.

2. Thawing gravel in winter was splashed against fenders and other parts of car and froze, and added considerable weight to the car.

3. The maximum safe speed on gravel during the winter and spring was frequently less than 45 m.p.h. whereas on concrete no difficulty was experienced in

maintaining 65 m.p.h. speed except for short periods when snow or ice was present on surface.

4. Frequently the maintainers when blading moist gravel, loosened the surface material which dried out and caused a large amount of gravel to be lost in the form of dust. Compacting the gravel by following the blade with a roller should provide a smooth firm surface on many sections of road on which the loss of gravel and the dust hazard would be small if maintained in this way.

5. Antiquated gravel road maintenance methods were used on certain sections of road in which the soil from the ditches was bladed over the entire road causing it to be muddy and slippery in wet weather or it was bladed in a windrow one or two feet high along the center of the road. In one case this windrow was so high that it was not safe for traffic to cross it.

6. The conditions under which poor visibility created a hazard were (1) when passing cars on loose dusty gravel roads, or (2) during rains on gravel when mud was splashed on the windshield which could not be entirely removed with the windshield wipers, and (3) during snow storms or when the road was covered with loose dry snow, the dry snow created a hazard similar to dust on a gravel road.

7. The concrete roads with a paved width of 18 ft. were too narrow under certain weather or traffic conditions. A rut frequently formed along the edge of the concrete or in certain locations the shoulder was washed away to a depth of several inches. When the shoulder was covered with soft snow, it was difficult to recognize this hazard and to tell where the slab ended and the shoulder began.

8. The usable width of the gravel roads was frequently greatly reduced because loose gravel in large windrows was placed along the center or the edge of road. When the car was forced into the loose gravel, there was a tendency to throw the car into a skid. If the loose material was on the edge of the road adjacent to a soft shoulder, it was fairly easy for the car to slide into the ditch before control could be regained.

9. Lip curbs and flumes on concrete roads backed up water on pavement to a depth of 3 inches or more during heavy rain storms and during thawing weather after heavy snowfall. Driving through water of this depth constituted a hazard. Also melting snow freezing suddenly creating patches of ice near flumes created an unexpected hazard.

10. Glare ice and sleet forming on concrete was more hazardous than the rough ice forming on the gravel surfaces.

11. The paint lines used to mark the center line of concrete pavements were found to be slippery when wet.

12. In general the skidding hazards on the gravel surfaces were greater than on the concrete surfaces. The maximum stopping rate on the gravel surfaces as indicated by the Tapley decelerometers mounted on each car was about 12 ft. per sec. per sec. While on the concrete surfaces a stopping rate of 18 ft. per sec. per sec. was readily obtained. When driving the cars on loose gravel or on sharp curves on the gravel route, even at moderate speeds, special care was required to keep the car under control to prevent skidding.

SUMMARY

The following summary of the important results obtained to date should be considered as tentative. For certain items of operating cost, such as the gasoline, oil, and tire costs, these results for 36,000 miles of driving with each car during the first year, may be considered as reasonably close to the final values which probably will be obtained after 60,000 to 65,000 miles of driving proposed on these surfaces.

1. The gasoline mileage for the first year of routine tests averaged 2 miles per gallon higher on the concrete than on the gravel surfaces over a wide range of speed.

2. The gravel cars used about twice as much oil as the concrete cars in terms of oil added.

3. The oil filter in Gravel Car No. 2 held the asphaltenes (gummy residue) in the oil to 0.22 per cent in 4,404 oil miles as compared to 1.02 per cent in 2,428 oil miles for Gravel Car No. 3 not equipped with an oil filter.

4. The tire wear on the gravel surfaces was practically double that on the concrete surfaces at 25 m.p.h., and at 50 m.p.h. the wear on gravel was about 50 per cent higher than on concrete.

5. For the first set of tires on the gravel cars 51 punctures were reported in 63,820 miles of driving, whereas only one puncture was reported in 72,290 miles for the tires on the concrete cars.

6. The brake linings and rear brake drums on the gravel cars were replaced due to excessive wear at the end of the first year or 36,200 miles of travel on the gravel route. Cylinder wear and bearing wear were considerably greater for the gravel cars than the concrete cars at the end of the first year.

7. The cost of gasoline, oil, and tires (based on tread wear) averaged ¼ cent per mile higher on gravel than on concrete over a wide range of speed. When tire wear and all maintenance expenditures for the first year are included, the costs on gravel averaged ½ cent per mile higher on gravel (average speed 35.8 m.p.h.) than on concrete (average speed 40.7 m.p.h.).

8. For all the cost items reported in (7), the use of an oil filter in this particular type of operation with changes in filter cartridges every 5,000 to 8,000 miles reduced these costs by about 1/10 cent per mile for the concrete cars and ⅛ cent per mile for the gravel cars.

9. On the basis of measurements with the Firestone riding comfort meter, the smoothest gravel section one mile in length tested at 40 m.p.h. was rougher than the roughest concrete section one mile in length tested at 40 m.p.h.

EFFECT OF TRAFFIC DELAYS ON GASOLINE CONSUMPTION

By A. J. Bone

Assistant Professor of Highway Engineering, Massachusetts Institute of Technology

SYNOPSIS

During the Spring of 1939, 130 determinations of gasoline consumption and travel time were made on 5.46 miles of streets in down-town Boston using a 1934 Ford Sedan for the test car. The runs were made at sufficient intervals to obtain a representative sample of traffic conditions throughout the busy hours (7 A.M. to 7 P.M.) of a typical weekday. Other tests were made on Sunday mornings when no congestion was present; the results of these latter tests when compared with the weekday tests served to indicate the increases in gasoline consumption caused by traffic delays.

These tests indicated that under ordinary conditions a driver will require about seven minutes to travel a mile; spending two and one-half minutes in traffic stops, one and one-half minute in first or second gear, and three minutes in high gear. Four to five complete stops will be required in each mile, averaging one-half minute per stop. These stops and other traffic interruptions will cause an increase of about 50 per cent in gasoline consumption over that required if no congestion were present. The average gasoline consumption for the test car was 12.1 miles per gallon in city traffic and 18.2 miles per gallon without traffic interference.

Assuming that all vehicles experience the same proportional increase in gasoline consumption as found for the test car, and with gasoline at 16 cents per gallon, the additional cost for gasoline caused by traffic delays amounts to about $18,000 per mile per year for the routes tested.

In July 1939, comparative tests were made with a 1937 Ford car on the Hutchinson River and Merritt Parkways in New York and Connecticut, and on the Boston Post Road (Route 1) paralleling these parkways. On a mile for mile basis the results of these tests showed an average gasoline consumption of 20.1 miles per gallon and on average speed of 38 miles per hour on the parkways compared with 18.4 miles per gallon and 25 miles per hour on the Boston Post Road. Each mile of driving on the parkways required three-quarters of a minute less time and 8.5 per cent less gasoline than required on the Post Road. The saving in gasoline cost amounted to about one-tenth cent per mile. When the proposed easterly connection between the Merritt Parkway and the Boston Post Road is completed, an estimated saving of 40 minutes in time and 3 cents in gasoline cost will be possible on the parkways between Pelham Manor, N. Y., and Milford, Conn. (55.5 miles) compared with the shorter, but frequently congested Post Road (53.5 miles).

A number of studies have been made of motor vehicle operating costs on different types of road surfaces on rural highways which have produced cost data useful for evaluating the economic advantages of rural highway improvement. In the more densely populated states a large portion of motor vehicle travel is performed on urban streets, where operating costs are increased by the congestion resulting from inadequate highway facilities.

The object of this investigation was to obtain some quantitative data with respect to the effect of traffic congestion on gasoline consumption and travel time with the idea in mind that such information, if more generally available, would assist in evaluating the advantages resulting from highway improvements for the purpose of expediting traffic in urban areas.

It is recognized that traffic conditions in cities are constantly changing from hour to hour and from street to street, and likewise that the operating characteristics of the different vehicles on the streets are not the same, so that no con-

clusions of general application can be drawn from the limited test results presented here. Nevertheless, it is felt that the results obtained with one car over a sufficient period of time to be representative of conditions in one locality would be of value in showing the relative gasoline cost and time consumed with and without traffic interference.

Part I of this report describes tests made in March and April, 1939, on several of the principal streets in the business district of Boston, Massachusetts. Part II relates to tests made in July, 1939, on the Hutchinson and Merritt Parkways in New York and Connecticut respectively, compared with similar runs on the Boston Post Road (Route 1) paralleling these parkways. In Part I a 1934 standard Ford Four-door Sedan was used. In Part II a 1937 Ford Club Cabriolet was used.

The first investigation originated as a bachelor's thesis submitted to the Massachusetts Institute of Technology by Messrs. W. F. Babcock and G. P. Morrison, to whom the author is indebted for much of the test data presented here. The second investigation was a research project of the Massachusetts Institute of Technology. Mr. Babcock prepared the apparatus and conducted the tests.

PART I

EFFECT OF TRAFFIC DELAYS ON GASOLINE CONSUMPTION AND TRAVEL TIME ON STREETS IN DOWNTOWN BOSTON

The street system of the business district of the city of Boston has come down from Colonial times. Most of the streets are narrow and crooked, and traffic flow is slow and torturous. Pedestrian traffic is so heavy as to be of major importance at many intersections. Figure 1 shows a plan of the city with the seven test routes indicated by heavy bands marked T-1, T-2, etc. These routes were selected to include typical main traffic routes across, around, and into the heart of the business district. Enough runs were made over each route to establish representative results for a normal week-day between the hours of 7 A.M. and 7 P.M.

Description of Routes

T-1 North Station to South Station—1.74 Miles

This route started at the corner of Nashua and Causeway Streets at the North Station, proceeded northeasterly along Causeway Street through Keany Square to Commercial Street; along Commercial Street to Atlantic Avenue, and then along Atlantic Avenue to Dewey Square at the South Station.

The route is used by a large volume of heavy commercial truck traffic during the day. Shortly before five o'clock, however, most of this heavy truck traffic disappears, causing a drop in traffic volume for a short time until the usual five-o'clock peak, when the street is again filled with traffic consisting principally of passenger vehicles.

The route is paved with a rather rough granite block surface, and a single-track railroad runs down the center of the street. This railroad extends over about one-half of the entire route and is used for transferring freight cars from waterfront sidings to the main-line freight terminal. When this railroad line is in operation considerable traffic delays result. Another cause for congestion is an abandoned elevated railway structure with two rows of supports in the middle portion of the roadway for its entire length. On the days when tests were made there was some road construction on the route, which may have increased the congestion, but the effect of this work was small compared with other causes for delay and had but slight influence on the results.



Figure 1. Location of Test Routes Down Town Boston, Massachusetts

The route has nine major intersections, six of which were controlled by officers at the time when the tests were made. There are also 33 entering streets which do not cross. There are no traffic lights on this route, but they were being installed at one intersection during the tests and are now in operation. The estimated daily traffic volume [1] for the 12 hours, 7 A.M. to 7 P.M., was 12,100 vehicles of all types. The width of roadway varied from 40 to 80 ft.

T-2 Congress Street—0.58 Miles

This route followed Congress Street from the west side of the Fort Point Channel to the rotary circle at Dock Square. The pavement was bituminous concrete in good condition.

Most of the traffic on the route was passenger vehicles, and much of the congestion was caused by pedestrians crossing at the numerous intersections. Pedestrian traffic was particularly heavy in the Post Office Square district.

There were seven full-crossing intersections and six entering streets which do not cross Congress Street. Five of the intersections were controlled by officers who were on duty the major part of the day. The remaining intersections had no form of traffic control.

The road was four lanes wide for the most part, but at times only three traffic lanes were available for traffic on account of parked cars. The estimated average daily traffic volume (7 A.M. to 7 P.M.) was 14,200 vehicles of all kinds.

T-3 Charles Street to Brattle Street—0.74 Miles

This route extended from the north side of Park Square on Charles Street, along Boylston Street to Tremont Street, and along Tremont Street to Brattle Street in Scollay Square.

[1] Traffic volumes were obtained from the Boston Traffic Survey.

The road surface was sheet asphalt in good condition. There were five streets that crossed and seven side streets. There were seven signal lights along the route supplemented by officer control, for the protection of pedestrians. This route entered the shopping district where pedestrian traffic was particularly heavy. The vehicular traffic was mostly passenger vehicles. The estimated average daily traffic for the 12 hours, 7 A.M. to 7 P.M., was 11,500 vehicles of all types.

T-4 Charles Street—0.43 Miles

This route started just south of the cross walk between the Boston Public Garden and the Boston Common on Charles Street and followed Charles Street northerly across Beacon Street to the traffic circle at the east end of the Longfellow Bridge.

The pavement was a fine texture, bituminous concrete in excellent condition. Although this street had a minimum width of four lanes, only three lanes were available for moving vehicles along much of the route on account of cars parked at the curb. There were four traffic lights on this route which were supplemented by officer control during peak hours. There were seven major intersections and one entering street which did not cross.

The estimated average daily traffic (7 A.M. to 7 P.M.) was 18,300 vehicles. During peak hours, the road became badly congested because of intermittent double parking which allowed movement in only two lanes.

T-5 Cambridge Street from Longfellow Bridge to Brattle Street—0.55 Miles

This route started at the traffic circle at the east end of Longfellow Bridge and followed Cambridge Street through Bowdoin Square to Brattle Street in Scollay Square.

The pavement consisted for the most part of granite block in good condition.

The route was six lanes wide except at the easterly end where it narrowed to four lanes and was further restricted by a subway station entrance in the center of Scollay Square. There were four traffic light installations, seven full intersections, and 11 side streets which did not cross. The estimated daily traffic (7 A.M. to 7 P.M.) was 16,900 vehicles of all types.

Most of the traffic on this route was passenger cars; the route is used extensively during the morning and evening peak hours by commuters from Cambridge and Somerville, and by shoppers during the middle of the day.

T-6 Adams Square to Keany Square—0.70 Miles

This route extended from the west side of Adams Square to Keany Square by way of Dock Square, North Street, Cross Street, Haymarket Square and North Washington Street, passing the entrance to the Sumner Vehicular Tunnel.

The pavement consisted of a variety of types, principally granite block and asphalt in good condition. The route was from three to five lanes wide and passed through two traffic circles. There were two traffic lights, three intersections controlled by officers, ten cross streets and seven side streets.

The daily traffic (7 A.M. to 7 P.M.) was 13,200 vehicles of all types. Much of the traffic was truck traffic and passenger car traffic entering or leaving the Sumner Tunnel (a vehicular tunnel under Boston Harbor connecting Boston and East Boston). From Haymarket Square to Keany Square much of the traffic was bound for or coming from Charlestown Bridge, connecting Boston and Charlestown.

T-7 North Station to Longfellow Bridge—0.72 Miles

This run started at the North Station and followed Nashua Street to the traffic circle at the east end of Craigie Bridge, thence along Charles Street to the traffic circle at the east end of Longfellow Bridge.

The route was 6-lanes wide divided by a 20-ft. center strip between the Longfellow and Craigie Bridges (0.40 miles) and a 6-lane roadway for the remaining distance. The pavement was asphaltic concrete in excellent condition. There were no traffic lights, no cross streets and no traffic officers except at one point for a short time during the evening peak hour. There were nine side streets and two traffic circles, one at the end of Longfellow Bridge and another at the end of the Craigie Bridge. Of the several routes tested this was the best adapted to motor traffic. The roadways were widened and the traffic circles constructed about ten years ago in an effort to relieve the intolerable traffic congestion existing in this district at that time.

The daily traffic (7 A.M. to 7 P.M.) was 24,000 vehicles of all types. There were six usable lanes most of the time; however, parked cars at the Massachusetts General Hospital (near Longfellow Bridge) cut down the usable lanes to four. Much of the traffic on this route used the Craigie and Longfellow Bridges, which connect Boston and Cambridge.

Test Car and Apparatus

The car used was a standard 1934 Ford Four-door Sedan in good running condition. During the tests the usual gasoline feed line to the fuel pump was disconnected and a gasoline measuring device substituted. The measuring device consisted of two 500 cubic centimeter glass burettes, both mounted on a stand which was fastened to the floor of the car at the right of the driver's seat. (See Fig. 2.) Copper tubes ran from the two burettes into opposite sides of a three-way Lunkenheimer valve. From the third opening in the three-way valve a

flexible tube ran into a special fitting which screwed into the fuel pump supplying the motor. The apparatus was embedded in sponge rubber to protect it from shocks and vibration.

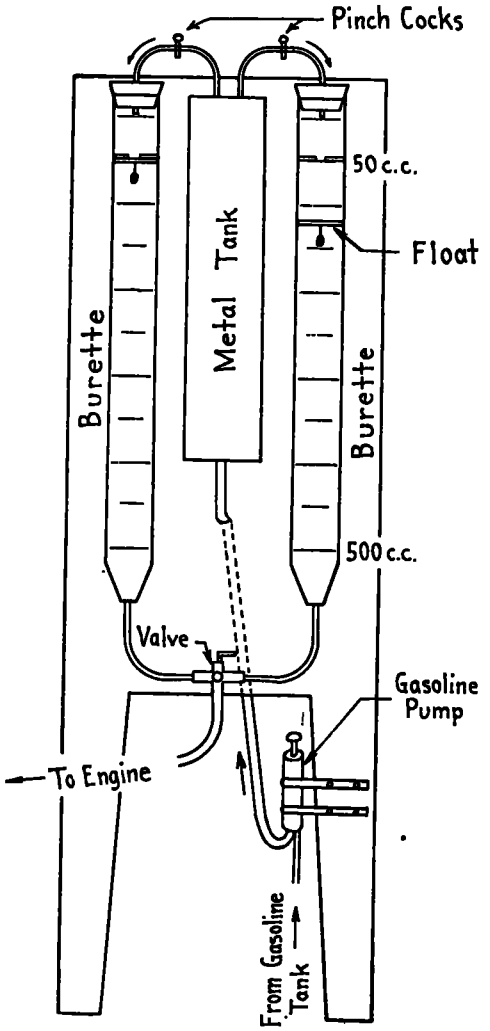


Figure 2. Gasoline Measuring Apparatus

The burettes were graduated in 5 c.c. divisions, and readings could be estimated to the nearest cubic centimeter. All measurements in cubic centimeters were converted into gallons and are so expressed throughout this report.

Gasoline could be pumped into either of the burettes by means of a hand pump connected to the feed line from the gasoline tank. Discs of cork and fine metal screens were floated on the surface of the gasoline in each burette; these served to damp out wave motions and made accurate readings possible even when the car was in motion.

One burette was sufficient to supply gasoline for the short runs made on the city streets of Boston. For these runs, the other burette was used as a reserve reservoir for bringing the car up to the speed of traffic at the beginning of the test and for reaching a convenient stopping place at the end of the test run. For the longer runs on the Merritt Parkway and Boston Post Road, the burettes were used alternately. The supply to the engine was shifted from one burette to the other by means of the three-way valve. This shift was accomplished while the car was in motion, without any appreciable error in the measurement of overall gasoline consumption. While one burette was feeding the engine, the other was replenished by means of the hand pump connected to the gasoline tank.

The gasoline level in each burette was brought to the 50 c.c. mark with the hand pump, and when connected to the engine, allowed to drop to the 500 c.c. mark before a shift was made to the other burette. In this way 450 c.c. were drawn alternately from each burette. It was found easier to set the floats at the levels desired than to attempt to read the level from the graduations. At the end of the test a reading of the last burette used was required to determine the final increment of gasoline taken from that burette.

The standard trip dial of the automobile cyclometer was used to determine the length of the test run. A cross-hair was placed over the front of the dial so that it could be read more accurately. The dial was ruled so that it could be read directly to the nearest half-tenth of

a mile, and estimated to the nearest one-hundredth of a mile.

Test Procedure in City Traffic

A schedule of tests was planned throughout the day so that different traffic conditions would be encountered and a fair average of daily conditions obtained. The car was driven with the traffic stream, imitating as far as possible the behavior of the average car. In most cases, complete round trips were made every half hour, but on some routes the time consumed permitted complete trips only once every hour. The average of runs in opposite directions gave average gasoline consumption and elapsed time for both directions of traffic flow and also balanced out the effect of grade and wind. Wind was of small consequence in these tests since the speed of travel was seldom over 25 miles per hour and the tests were made in relatively quiet weather.

Another series of runs were made over the same routes early Sunday mornings when the streets were nearly clear of cars and the runs could be made without interruption on account of other traffic. A comparison between these runs and those made during the week showed the effect of congestion both on travel time and on fuel consumption.

Before starting a test run the level of the gasoline in the test burette was brought to the 50 c.c. mark, and the reserve burette filled to a convenient level. An initial distance reading was taken on the cyclometer, but with the trip mileage recorder disengaged. The car was then started and brought up to speed using gasoline from the reserve burette. When the starting point of the test route was reached the gasoline supply to motor was shifted to the measuring burette and the trip cyclometer thrown in. At the end of the test route the gasoline supply was shifted to the reserve burette and the trip mileage disengaged. The car was then driven to a convenient stopping place and the gasoline level in the measuring burette recorded. The difference in readings gave the c.c. of gasoline used during the test run. The difference in trip mileages gave the distance covered. The distances were verified by scaling from a map of the street layout.

Three stop watch recordings were made. The first stop watch, operated by the driver, was used to record the total elapsed time for the run. The second, also operated by the driver, was used to record each interval of time that the engine was idling (*i.e.*, the car stopped in a traffic delay); the aggregate of these latter readings gave the cumulative time that car was not in motion. An observer, sitting beside the driver, operated a third stop watch to record the cumulative time that the car was operated in either of the low gears (first and second). Stop watch readings were used to compute the total time, time in high gear, time in first and second gears, and time that the motor was idling. The observer recorded the number of complete stops that were made on the run, and also any unusual conditions encountered during the run.

Variation of Gasoline Consumption with Speed on the Open Road

In order to compare gasoline consumption on the open road with that obtained in city traffic, the upper curve in Figure 3 was developed showing the gasoline consumption characteristics of the test car over a speed range of 5 to 60 miles per hour. This curve was plotted from the results of tests on a 4-lane concrete highway. The plotted points are the average of runs in each direction over a mile course. This curve represents the performance of the test car under "ideal" conditions without traffic or other interference of any kind.

The middle curve is the same as the lower part of the upper curve, but with the scale of miles per hour increased five times. The lower curve, to the same scale as the middle curve, shows the average miles per gallon obtained at different average speeds in city traffic under stop and go conditions. The difference between these curves indicates the decrease in gasoline mileage caused by traffic delays.

Results Obtained with Test Car

The results of test runs are summarized in Table 1. The first group of items shows conditions influencing traffic flow along the routes; such as the number of street crossings, number of control points (signals or officers), available traffic lanes and average daily traffic volumes. The number of available traffic lanes varied from street to street and from time to time depending upon the

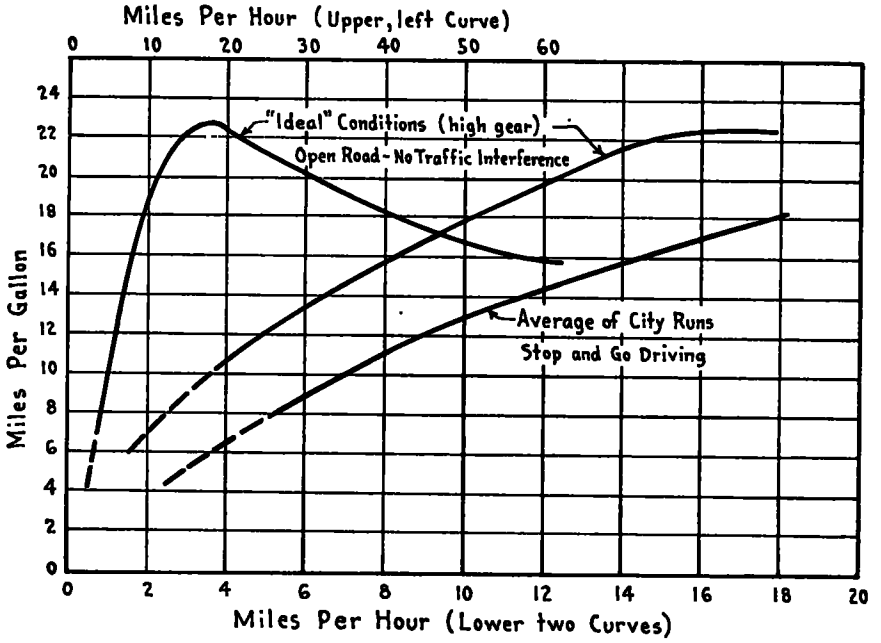


Figure 3. Relation between Gasoline Consumption and Speed for Test Car—1934 Ford

The lower curves extend only to about 18 miles per hour since higher average speeds were not possible in city traffic. On the open road higher speeds would prevail and comparisons can then be drawn between the upper and lower curves. For example, the gasoline consumption rate at 40 miles per hour on the open road for the test car was 18.0 miles per gallon (upper curve) and for 10 miles per hour in city traffic it was 12.9 miles per gallon (lower curve).

width of the pavement and the extent to which street space was taken up with parked cars.

The second group of items in Table 1 indicates the nature of the delays encountered. The number of traffic stops is the number of times the car had to be brought to a complete stop. These stops usually occurred at signal lights or at officer control points, but occasionally were caused by traffic interference alone. The average length of stop was about

TABLE 1

DESCRIPTION OF ROUTES—ANALYSIS OF DELAYS—COMPARISON OF AVERAGE RATE OF GASOLINE CONSUMPTION FOR TEST CAR WITH AND WITHOUT TRAFFIC INTERFERENCE

Route No.	T-1	T-2	T-3	T-4	T-5	T-6	T-7	All routes
DESCRIPTION OF ROUTE								
Length (miles)	1.74	0.58	0.74	0.43	0.55	0.70	0.72	5.46
Streets crossing	9	7	5	7	7	10	2[1]	47
Streets entering—not crossing	33	6	7	1	11	7	9	74
Number of traffic control points	6	5	7	4	4	2	0	28
Lanes available for moving traffic	4 to 6	4	3 to 4	3	4 to 6	3 to 5	4 to 6[2]	....
Average daily traffic (7 A. M. to 7 P. M.)	12,100	14,200	11,550	18,300	16,900	13,200	24,000	14,940
ANALYSIS OF DELAYS								
Traffic stops necessary per trip	4.5	3.6	8.1	2.6	1.8	3.7	0.5	24.8
Average time per stop (seconds)	22.7	13.0	39.2	31.0	27.3	38.5	4.7	31.8
Percent of time stopped	18.5	34.9	52.6	41.5	26.2	39.6	1.6	34.1
Percent of time in low gears	20.7	24.5	25.4	16.8	14.6	20.9	3.4	20.5
Percent of time in high gears	60.8	40.6	22.0	41.7	59.2	39.5	95.0	45.4
AVERAGE OVER-ALL SPEED								
Miles per hour	11.3	7.7	4.4	8.0	10.5	7.0	18.1	8.5
AVERAGE GASOLINE CONSUMPTION								
Miles per gallon in traffic	14.3	10.5	7.2	12.2	14.9	11.1	19.2	12.1
Miles per gallon without traffic (Sunday morning runs)	18.7	17.3	15.7	20.4	18.1	18.2	20.0	18.2

[1] Traffic circles. [2] Divided roadway, 3 lanes each side of 20 ft. grass strip for .4 mi.

T-1 Atlantic Ave. and Commercial St., North Station to South Station.
T-2 Congress St., Fort Point Channel to Dock Sq.
T-3 Tremont and Boylston Sts., Scollay Sq. to Park Sq.
T-4 Charles St., Boylston St. to Longfellow Bridge.
T-5 Cambridge St., Longfellow Bridge to Scollay Sq.
T-6 Adams Sq., Dock Sq., Cross and North Washington Sts.
T-7 Nashua and Charles Sts., North Station to Longfellow Bridge.

30 sec., but individual stops varied in length from a few seconds to several minutes. The percentage of the total time consumed in traffic stops is an indication of the seriousness of these delays. They accounted for 52.6 per cent of the time on the most congested street (T-3) and only 1.6 per cent of the time on the least congested street (T-7).

At times the speed of traffic was so slow that the test car had to be operated in first or second gear. The time spent in these gears is shown in percentage of elapsed time for each route. It amounted to 25.4 per cent on the most congested route (T-3) and 3.4 per cent on the least congested route (T-7).

The average over-all speed of the test car is given for each route. The range was from 4.4 miles per hour for the most congested route (T-3) to 18.1 miles per hour for the least congested route (T-7).

The lower group of items in Table 1 shows a comparison between average gasoline consumption obtained by the test car with and without traffic interference. The values for "miles per gallon in traffic" are averages of the results obtained with the test car at half-hour or hourly intervals during a normal week day. The values recorded for "miles per gallon without traffic" were obtained from Sunday morning runs when very few vehicles were on the streets. These Sunday morning runs do not represent wholly unrestricted traffic flow, however, since traffic signals were in operation on some streets, particularly on Tremont Street (T-3) where the Sunday morning average gasoline consumption was only 15.7 miles per gallon.

The average speed of travel on the Sunday morning tests was about 20 miles per hour. The street system of Boston is so irregular and blind intersections so prevalent that higher average speeds were not justified from the safety standpoint.

If new highway facilities should be provided to relieve traffic congestion in the heart of the city, the vehicles that made use of these facilities would travel faster than is possible on the present street system and enjoy greater savings in time than shown in this report, as well as enjoying a substantial saving in gasoline consumption. The vehicles still using the street system for local business purposes would benefit by the reduction in traffic delays resulting from the reduction in traffic volume on these streets because of the diversion of non-local traffic to the new facility.

In comparing the results shown in Table 1, weight should be given to the fact that conditions on Tremont Street (T-3) were more congested than normal. The test runs were made on a Saturday morning one week before Easter. This street skirts the retail department and clothing store district so that Easter shopping crowds added to the congestion. At best, however, the route is a slow one to traverse since it has many signal lights which must accommodate large volumes of turning traffic and pedestrian traffic as well as through traffic. The northerly end of the street is narrow and tortuous; the traveled way is restricted to only two lanes in places due to cars and trucks stopping at the curb. Results for this route are more typical of conditions that occur periodically rather than those prevailing on every normal week day.

Additional Gasoline Cost Resulting from Delays

In order to measure accurately the additional gasoline cost incurred by all traffic on account of traffic delays, it would be necessary to know the gasoline consumption characteristics of each of the many different types of vehicles using the streets, and also the distribution of these vehicles in the traffic stream. This

would require a far more extensive study than that presented here. From the limited data available from this study, an estimate has been made of the increased gasoline cost for all traffic based on the gasoline consumption characteristics of the test car. Obviously the test car is not representative of all traffic; the newer cars will be more efficient in their use of gasoline, whereas the older cars and the heavy commercial vehicles will have lower mileage rates of gasoline consumption.

What is desired, however, is the difference in gasoline cost with and without congestion. Although the rates of gasoline consumption obtained by different vehicles may vary through a wide range, the difference in costs, with and without delays, will be less variable. The results of this study, although based upon but one vehicle, are indicative, at least, of the extra cost incurred by all traffic on account of traffic delays. This estimate is developed in Table 2.

A weighted average value for gasoline consumption throughout the 12-hour test period (7 A.M. to 7 P.M.) was obtained for each route by weighting the results of each run in proportion to the total number of vehicles using that route during the hour when the test was made. This procedure resulted in weighted average rates of gasoline consumption which differ slightly from those in Table 1.

These weighted rates of gasoline consumption compared with those obtained without delays produced the percentages of increase in fuel consumption given in Table 2 for each of the seven routes tested. The increase in consumption ranges from 3.1 per cent for the least congested route to 118.0 per cent for the most congested route.

The increase in gasoline consumption was translated into cost by finding the difference in the gallons of gasoline consumed over the route, with and without delays, and evaluating this quantity at the rate of 16 cents per gallon; the prevailing retail price in the Boston district when the tests were made.

The estimated costs of traffic delays for each route are shown in Table 2 as follows: per day of 12 hours (7 A.M. to 7 P.M.), per year of 300 days (excluding Sundays and holidays) and per mile per year of 300 days. On the latter basis the annual gasoline cost for traffic delays ranges from $1,750 per mile for the least congested street to $41,600 per mile for the most congested.

Discussion of Results of Table 2

The results in Table 2 are conservative to the extent that they cover only the 12 hours (7 A.M. to 7 P.M.). These are the most congested hours of the day, but delays also occur during the evening hours, particularly where signal lights are in continuous operation.

The tests on Route T-3, which showed the greatest increase in cost, were made at a particularly unfavorable time, namely, during the Easter shopping season. The annual estimate for this route is probably too high. On the other hand the results for the other routes are probably too low since they were made during the Spring months when city traffic is somewhat below the annual average.

The small increase in cost for Route T-7 is due to the more liberal width of the streets on this route, greater traffic capacity and the absence of restrictive cross traffic. The two major intersections on this route are traffic circles. It is significant to note that the streets comprising this route were widened and the traffic circles constructed within the last 10 years. Before these improvements were made, traffic conditions were as bad as any found in these tests. The difference between the high cost of delays on the other streets and the low cost of delays for this route serves to show the

TABLE 2

ESTIMATED INCREASE IN GASOLINE COST CAUSED BY TRAFFIC DELAYS

Gasoline at 16 Cents Per Gallon

Route No.	T-1	T-2	T-3	T-4	T-5	T-6	T-7	All routes
Length (miles)	1.74	0.58	0.74	0.43	0.55	0.70	0.72	5.46
Weighted average gasoline consumption *with* traffic delays (miles per gallon)	14.0	10.5	7.2	11.8	14.9	11.3	19.4	12.5
Gasoline consumption *without* traffic delays (miles per gallon)	18 7	17.3	15.7	20.4	18.1	18.2	20.0	18 5
Percent increase in gasoline consumption caused by traffic delays	33.6%	64.8%	118.0%	72.9%	21.5%	61.1%	3.1%	48.0%
Average traffic volume—total vehicles per weekday (7 A. M. to 7 P. M.)	12,100	14,200	11,550	18,300	16,900	13,200	24,000	14,940
Estimated cost of traffic delays per day (7 A. M. to 7 P. M.)	$59 80	$49.30	$102.50	$45 20	$17.60	$49.50	$4.20	$328.10
Estimated cost of traffic delays per year (300 days)	$17,940	$14,790	$30,750	$13,560	$5,280	$14,850	$1,260	$98,430
Estimated cost of traffic delays per year per mile of route	$10,300	$25,500	$41,600	$31,500	$9,600	$21,200	$1,750	$18,000

dividends in gasoline cost savings which have resulted from the investment made in these improvements.

Variation of Gasoline Consumption with Traffic Volume

In order to study the influence of traffic volume on gasoline consumption, charts were prepared showing to the same scale the hourly variation in gasoline consumption compared with the hourly variation in traffic volume. Both gasoline consumption and traffic volumes were expressed in percentage of the value for the maximum hour, which was made equal to 100 per cent in each case. Two of the charts so obtained are described below.

Figure 4 shows the comparison obtained for Route T-1, wherein the curve of gasoline consumption followed the curve of traffic volume rather closely. The maximum in each case occurred at 5 P.M., the usual peak hour for traffic flow. On this route the delays were not caused so much by cross traffic at intersections as by interference between vehicles using the route. For the most part the route followed the water front, so that streets could enter on only one side; however, there were many driveways to wharves and warehouses on the ocean side. Much of the traffic interference on this route resulted from the movement of trucks to and from these wharves and warehouses. As the traffic increased, more cars were delayed and the delays became more frequent and of longer duration. Hence the gasoline consumption increased in about the same proportion as traffic volume. This was not typical of all streets, however.

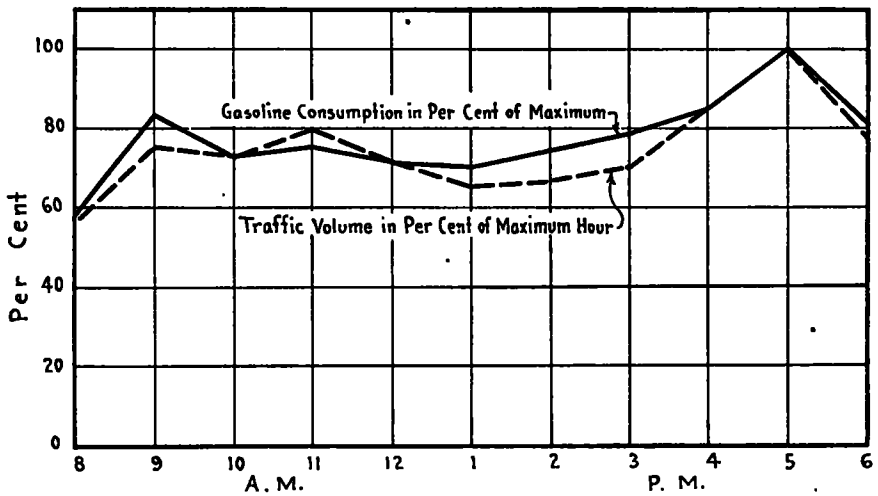


Figure 4. Hourly Variation in Gasoline Consumption and Traffic Volume Expressed in Per Cent of Maximum Route T-1 North Station to South Station via Atlantic Avenue and Commercial St., Boston, Mass.

Figure 5 shows comparative curves of gasoline consumption and traffic volume for Route T-2. These curves do not follow each other closely, particularly at peak hours. During the morning peak from 7:30 to 9:30 A.M. traffic volumes were high but there was no corresponding increase in gasoline consumption. Conversely the peak in gasoline consumption at 5 P.M. was accompanied by an actual drop in traffic volume. These

results may be traced to the characteristics of the traffic flow on this street.

During the morning peak hours the traffic volume is high but evenly distributed within the peak hours. There is a steady influx of cars to the district during these hours; many of these cars are parked for the day in parking lots along the route. Since the traffic is distributed during the morning hours, the street is able to handle it reasonably well without unusual delays. At 5 P.M. an entirely different set of conditions exists.

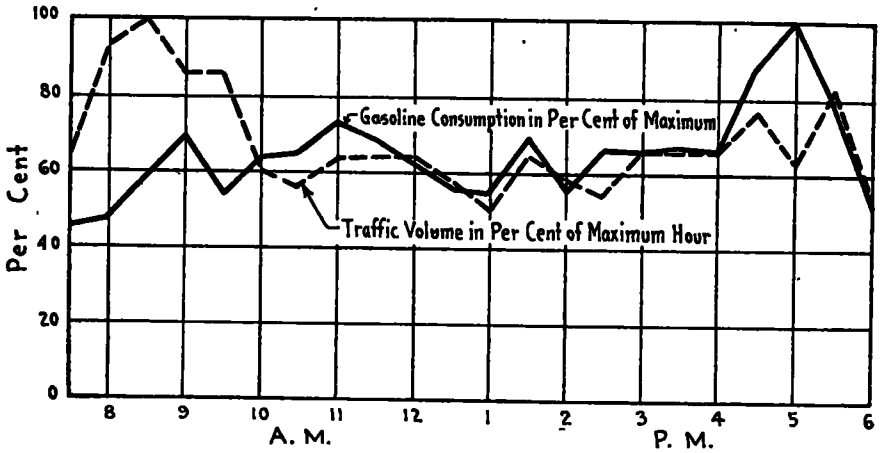


Figure 5. Hourly Variation in Gasoline Consumption and Traffic Volume Expressed in Per Cent of Maximum Route T-2 Congress Street-Fort Point Channel to Dock Square, Boston, Mass.

All cars attempt to leave the city at about the same time; they flood into the streets from all directions and cross traffic is particularly heavy. Although the demand on the route was heavy at this time, congestion developed to such an extent that only a limited number of cars could get through and the volume of cars passing through the route at 5 P.M. was actually less than that at 4:30 P.M.

Time Lost in Traffic Delays

Before an estimate can be made of the amount of time lost in traffic delays, a representative speed must be adopted for traffic without congestion. On the routes covered in this study the average speed would not be very great even if the delays which now occur could be eliminated. Many of the streets are crooked and the view is obstructed at nearly every corner. From a safety standpoint top speeds over 25 miles per hour are undesirable, and an average speed of 20 miles per hour is about the highest that should be considered as reasonable, even without congestion.

If new facilities of the express highway type were provided to relieve the traffic congestion found in this study, the average speed on such roads would be greater than 20 miles per hour. For those drivers who would use these facilities in place of the present street system, a greater saving of time would be possible. The saving in gasoline cost would be less than indicated in Table 2, however, since fuel consumption increases with speed. For these faster drivers a portion of the potential saving in gasoline cost would be exchanged for a saving in time.

Table 3 shows for each of the test routes the time lost in traffic delays causing the average speed to be reduced below 20 miles per hour. The speeds obtained with the test car in different

TABLE 3

ESTIMATE OF TIME LOST IN TRAFFIC DELAYS CAUSING AVERAGE SPEED OF TRAFFIC TO BE REDUCED BELOW 20 MILES PER HOUR

Route No.	T-1	T-2	T-3	T-4	T-5	T-6	T-7	All routes
Length of route (miles)	1.74	0.58	0.74	0 43	0 55	0 70	0.72	5.46
Weighted average speed in city traffic (miles per hour)	10.9	7 7	4 4	7.6	10 5	7.2	18.1	8.4
Time required to travel route at above speed (minutes)	9 6	4.5	10.1	3 4	3.1	5.8	2.4	38.9
Time that would be required at 20 m.p.h. (minutes)	5.2	1.7	2.2	1.3	1 6	2.1	2.2	16.3
Time lost per vehicle in delays causing an average speed of less than 20 m.p.h. (minutes)	4.4	2.8	7 9	2.1	1 5	3.7	0.2	22.6
Daily traffic using route (7 A. M. to 7 P. M.)	12,100	14,200	11,550	18,300	16,900	13,200	24,000	14,940
Time lost daily (7 A. M. to 7 P. M.) by all vehicles using route (in hours)	889	663	1,520	641	423	813	80	5,029
Total time lost by all vehicles per year of 300 twelve-hour days (in hours)	267,000	199,000	456,000	192,000	127,000	244,000	24,000	1,509,000
Hours lost per year per mile from above	153,000	343,000	671,000	447,000	231,000	348,000	33,000	277,000

hours on a given route were weighted according to the volume of traffic using the street at the time when a test was being made. The weighted average speed obtained in this manner was considered to be representative of the average speed of all traffic using the route during the hours 7 A.M. to 7 P.M. From this average speed and the length of route, the time required to cover the route under normal, weekday traffic conditions was computed. The difference between this interval of time and that which would have been required at 20 miles per hour gave the time lost per vehicle on account of traffic delays. The time lost per vehicle multiplied by the average daily traffic (7 A.M. to 7 P.M.) gave the time lost per 12-hr. day. This estimate was expanded to a year by considering that traffic conditions on 300 days of the year would be substantially the same as found by the test car on a representative day. A year of 300 days was used rather than one of 365 days because the streets tested were the type that are heavily traveled on weekdays and lightly traveled on Sundays and holidays.

The estimate of time is a very rough one since it has been expanded greatly from limited data, but it is probably an understatement rather than an overstatement of the time lost, because in does not include delays which occur during the night hours from 7 P.M. to 7 A.M., nor does it include any delays which may be encountered occasionally on Sundays and holidays.

The estimated time lost annually on the above basis varies from 24,000 hours for the least congested route (T-7) to 456,000 hours for the most congested route (T-3).

Summary of Results and Conclusions for All Test Routes

The results obtained with the test car on the seven test routes aggregating 5.46 miles and including 130 observations are summarized in Table 4. From these results the following general conclusions may be drawn regarding Boston traffic conditions as found on the seven routes tested:

Under ordinary weekday conditions a driver will require about seven minutes to travel a mile. Of this time roughly two and one-half minutes will be spent in traffic stops, one and one-half minutes traveling in first or second gear, and three minutes traveling in high gear. In traveling a mile a driver will be required to make from four to five complete stops, spending on an average of one-half minute in each stop. These stops and other traffic interruptions will cause an increase of about 50 per cent in gasoline consumption over that required if no congestion were present.

TABLE 4

SUMMARY OF RESULTS FOR ALL TEST ROUTES (5.46 MILES)

TEST CAR, 1934 FORD SEDAN

Traffic Stops Required per Mile	=	4.5
Per Cent of Time Stopped	=	34.1%
Average Time per Stop in Seconds	=	32
Per Cent of Time in Low Gears	=	20.5%
Per Cent of Time in High Gear	=	45.4%
Average Speed of Test Car in Miles per Hour in City Traffic	=	8.5
Average Gasoline Consumption without Congestion in Miles per Gallon	=	18.2
Average Gasoline Consumption in City Traffic in Miles per Gallon	=	12.1
Percentage Increase in Gasoline Consumption caused by Traffic Delays	=	50.4%
Average Traffic Volume per Twelve-Hour Day (7 A.M. to 7 P.M.)	=	14,940
Estimated Additional Cost of Gasoline due to Traffic Delays per Year per Mile for all Traffic	=	$18,000
Estimated Time Lost by All Traffic because of Delays causing the Average Speed to be Less than 20 Miles per Hour in Hours per Year per Mile	=	277,000

The average daily traffic volume on the streets tested was about 14,940 vehicles of all kinds in 12 hours (7 A.M. to 7 P.M.). Assuming that all these vehicles experienced the same proportional increase in gasoline consumption as found for the test car, and with gasoline at 16 cents per gallon, the additional cost for gasoline caused by traffic congestion amounts to about $18,000 per mile per year.

Compared with the time that would be required at an average speed of 20 miles per hour, traffic delays cause an aggregate loss of time of about 277,000 hours per mile per year.

PART II

GASOLINE CONSUMPTION AND TIME STUDIES ON THE HUTCHINSON RIVER PARKWAY, N. Y., THE MERRITT PARKWAY, CONN., AND THE BOSTON POST ROAD, ROUTE I, PARALLELING THESE PARKWAYS

During the first two weeks of July 1939, a number of tests similar to those described in Part I were conducted on the Hutchinson River and Merritt Parkways and on the Boston Post Road which parallels these parkways. The purpose of these tests was to compare gasoline consumption and travel time on the unobstructed parkways with results obtained on the Boston Post Road, which passes through many cities and towns.

The parkways were built and are now being extended at considerable expense to relieve traffic congestion on the Boston Post Road, and to provide easy access to New York City from the northeast. Figure 6 shows the location of the roads covered in this study and proposed extension of the parkway system across Connecticut. The use of the parkway is restricted to passenger cars, and a 10-cent toll is charged on the Merritt Parkway from Westport to the New York state line. This study was not undertaken for the purpose of justifying or disproving the principle of levying tolls, and no discussion of the toll is included in this report.

The study was intended, however, to develop some data on the more tangible advantages gained by motorists using the parkways in preference to the Post Road; namely, savings in gasoline consumption and savings in time. There are many other less tangible but no less real advantages offered by the parkways, such as the satisfaction of being able to drive safely and swiftly through a pleasant countryside rather than the nerve-racking strain of bucking a seemingly endless stream of traffic. The photographs in Figure 7 show typical scenes on the Boston Post Road and Merritt Parkway.

Description of Test Routes

Hutchinson River Parkway: Test runs were made on the Hutchinson River Parkway between the overpass at the Boston Post Road in Pelham Manor, N. Y., and the New York-Connecticut state line at Route 120A where the Merritt Parkway begins—a distance of 14.74 miles. The roadway is four lanes of cement concrete and is undivided except for about 0.3 mile at the easterly end where the road is two lanes each way separated by a 10-ft. grass strip. There are no crossings at grade, but access may be had to the parkway at 11 grade separations by means of right turns to or from the outside lanes. There is one traffic circle at the junction with the Cross Country Parkway. The annual traffic volume is about 15,000 passenger cars per day.

Merritt Parkway: This parkway is the Connecticut extension of the Hutchinson River Parkway. Test runs were made between the New York state line and the junction with Route 33 in Westport—a distance of 21.17 miles. In 1939 the parkway was open as far as Route 33, where traffic was detoured to the Post

Road via this route. Westbound traffic on the Post Road may enter the parkway via Route 57 from Westport, which brings such traffic on to the parkway about 0.3 mile east of Route 33. In order to keep the length of test runs the same for both directions of travel, the Route 33 junction was used as the terminus for both eastbound and westbound runs on the parkway.

In 1940 the Merritt Parkway will be open to a point beyond Bridgeport (see Fig. 6). Although no tests could be made on this uncompleted section, an estimate has been made of the probable gasoline consumption and travel time on this stretch of parkway between Route 33 and the proposed future junction with the Post Road at Milford. This was done for the purpose of comparing runs on the Boston Post Road from Westport to Milford with conditions that will prevail after the parkway is open.

The Merritt Parkway has two 26-ft. cement concrete roadways separated by a 21-ft. grass strip (except under certain bridges where the center strip narrows to 3 ft.). There are no important crossings at grade. Access to the parkway is provided by right turns at nine grade separations between the New York state line and Westport. An additional outer lane is provided for acceleration and deceleration at ramps to overpassed or underpassed highways. The terrain is rolling country so that the highway is rising or falling most of the way. The maximum grade is about six per cent and the sharpest curve 7 deg. The right-of-way is 300 ft. wide and marginal access is prohibited except at certain grade separations.

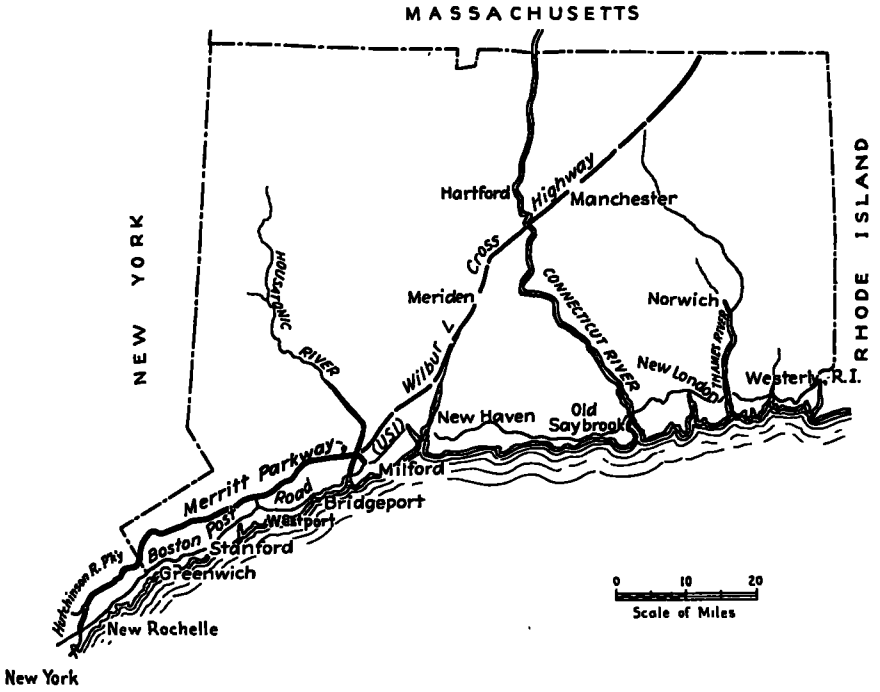


Figure 6. Location of Hutchinson River and Merritt Parkways Showing Proposed Wilbur L. Cross Highway

Boston Post Road

Separate recordings were made for three sections of the Post Road in order that comparisons could be drawn with the parkways between connecting routes joining the parkways and the Post Road. These connecting routes were 120A in Port Chester, Route 33 in Westport, and Route 113 in Stratford.

Hutchinson River Parkway overpass in Pelham Manor, N. Y., to Route 120A junction in Port Chester, near the Connecticut line—11.84 miles: This section of the route passes through the main streets of the city of New Rochelle, N. Y. (54,000 population) and through the village of Port Chester (22,662 population). In the latter place alternate routes

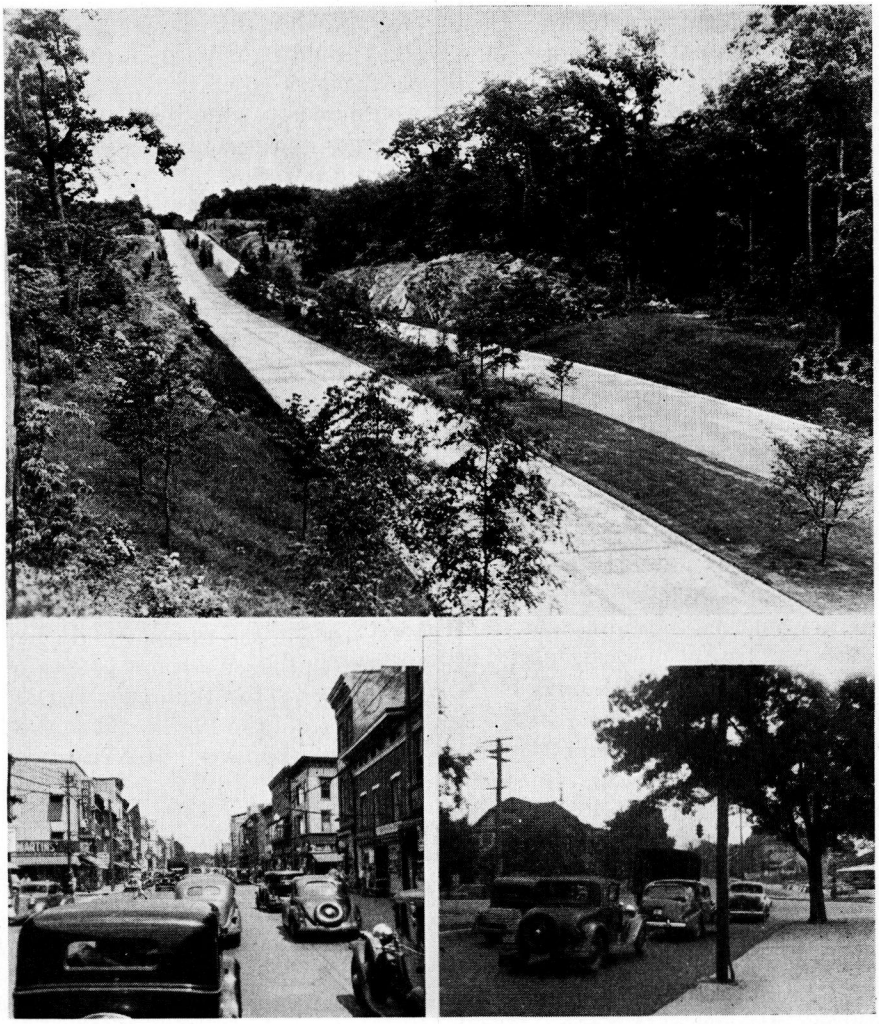


Figure 7. Views of Merritt Parkway (upper) and Boston Post Road (lower) in Port Chester (left) and Bridgeport (right)

are marked for eastbound and westbound traffic. The eastbound route skirts the business district and the westbound route follows the principal business thoroughfares. There is a slight difference in the length of the routes; eastbound 11.76 miles and westbound 11.92 miles. Along this section there are 126 intersecting streets and 36 traffic control points, of which 32 are signal lights.

Route 120 junction in Port Chester, N. Y., to Route 33 junction in Westport —20.96 miles: This section passes through the centers of Greenwich (33,-112), Stamford (46,346), Darien (6,951) and Norwalk (36,019). Alternate routes are provided through some of these places so that the eastbound mileage is 21.02 and the westbound mileage 20.90. There are 220 intersecting streets and 62 traffic control points, of which 54 are signal lights.

Route 33 junction in Westport to Route 113 junction in Stratford—15.12 miles: This section passes through Fairfield (17,218) and Bridgeport (146,716). The route is marked to by-pass the principal business district of Bridgeport, but this by-pass is still well within the built-up part of the city. Along this section are 147 intersecting streets and 34 traffic control points, of which 30 are signal lights.

At the time the tests were made it was anticipated that the Merritt Parkway would connect with the Post Road near Route 113 in Stratford, and test runs on the Post Road were stopped at the junction with Route 113. Present plans of the Connecticut Highway Department call for extending the Parkway across the Housatonic River to connect with the Post Road in Milford, as shown in Figure 6. An estimate has been made of the gasoline consumption and travel time for the Post Road between Route 113 in Stratford and the proposed parkway connection in Milford—5.55 miles.

The pavement of the Boston Post Road is 4-lane concrete between cities and of varying width within towns and cities. In places the traffic stream is reduced to one lane in each direction on account of parked cars. There is very little of the Post Road within the limits of this study that could be classed as rural. Between towns and cities it is bordered almost continuously with roadside restaurants, garages and gasoline stations.

Connecting Routes

Test runs were made over three routes connecting the parkways and the Post Road so that "closed circuits" could be made between portions of the parkways and the Post Road.

Route 120A was run between the Hutchinson River Parkway and Port Chester, N. Y., a distance of 2.50 miles.

Route 33 was run between the Merritt Parkway and the Post Road in Westport, Conn., a distance of 1.97 miles. This route is marked as the exit for eastbound traffic on the parkway.

Route 57 was run northerly from the Post Road in Westport to the Merritt Parkway. This route covered the approach from the Post Road to the Parkway in the westerly direction—1.70 miles.

Test Car and Test Procedure.

The car used was a 1937 Ford convertible Club Cabriolet. The gasoline measuring device was the same as shown in Figure 2.

The test runs differed only in length from those made in Boston traffic. The runs were from 15 to 20 miles long and required several shifts from one measuring burette to the other in order to obtain all the gasoline required for the run. On the Boston Post Road runs, stop-watch readings were kept of elapsed time and of time stopped. On the parkways no traffic interference was encountered. The

only stops required were to pay the toll on the Merritt Parkway.

An equal number of runs was made in each direction to balance the effect of grades and wind. Tests were run during different hours of the day and on different days of the week in order to obtain a representative sample of gasoline consumption and travel time on each route. All tests were run between the hours of 8 A.M. and 6 P.M. No tests were made at night. Some tests were made on the Fourth of July week-end in order to observe the effect of heavy traffic flow as well as normal summer weekday conditions.

Traffic Volumes

The traffic on the Boston Post Road during the days when tests were made was recorded by automatic counters installed by the State-Wide Planning Survey at Westport as follows:

	Eastbound	Westbound	Total
Monday, July 3	12,503	12,474	24,977
Tuesday, July 4 (holiday)	12,704	13,746	26,450
Wednesday, July 5	11,668	14,528	26,196
Thursday, July 6	10,420	10,947	21,367
Friday, July 7	12,074	11,402	23,476
Saturday, July 8	13,325	11,480	24,805
Monday, July 10	10,765	11,929	22,694
Tuesday, July 11	10,292	10,965	21,257
Wednesday, July 12	10,361	10,768	21,129
Thursday, July 13	10,542	11,087	21,629

The character of traffic using the Boston Post Road, as indicated by an 8-hour count (2 P.M. to 10 P.M.) made by the State-Wide Planning Survey at Greenwich on July 12, 1939, was as follows:

		%
Passenger Cars	6,397	82.3
Light Trucks	444	5.7
Medium Trucks	427	5.5
Heavy Trucks	226	2.9
Tractor Semi-trailers	220	2.8
Busses	61	0.8
	7,775	100.0

The Post Road offers the only direct connection between New York City and New England for commercial traffic. It carries, therefore, an unusual concentration of heavy, long-haul trucks and trailer trucks. These large vehicles add to the congestion in urban districts and slow down traffic on grades. Although the Post Road would not be classed as hilly, there are frequent grades of 4 or 5 per cent which often slow these trucks down to 5 or 10 miles per hour.

Daily traffic on the Merritt Parkway during the days when test runs were made was recorded from toll collections, as follows:

Monday,	July	3	18,269
Tuesday,	July	4	23,613
Wednesday,	July	5	19,307
Thursday,	July	6	12,564
Friday,	July	7	15,406
Saturday,	July	8	20,123
Sunday,	July	9	26,430
Monday,	July	10	15,342
Tuesday,	July	11	12,849
Wednesday,	July	12	12,642
Thursday,	July	13	12,754

The average annual traffic on the Boston Post Road at Greenwich amounts to about 16,000 per day. On the Merritt Parkway the yearly average day is estimated at about 13,700 vehicles. When the Merritt Parkway was opened, the traffic on the Boston Post Road at Greenwich diminished about 6,600 vehicles; approximately 7,100 vehicles were attracted to the Parkway from sources other than the Boston Post Road. Some of this traffic was attracted from Route 6, which previous to the construction of the Parkway, offered the only practical sub-

stitute for the Post Road for New York-New England through traffic.

Results

The average results for each set of runs are summarized in Table 5 which gives the length of each route, the interruptions encountered, time required, average speed, gasoline consumption and gasoline cost. The latter was computed at 16.5 cents per gallon, the prevailing retail price of gasoline in the district.

In the 48 miles covered on the Post Road, 41 complete stops were required, a little less than one stop per mile. Nearly all of these stops were at traffic lights, of which there were 116 along the route. The test car was stopped at about one out of every three lights. The average length of stop was 23 sec.

The average speed derived from the elapsed time and the distance covered was 25.0 miles per hour. As the test car was driven at the average pace of all traffic, this represents the speed of the average car. Some drivers may be able to make better time by more "flashy" driving, and others of a more cautious nature will have slower average speeds. The speed on the Post Road is primarily limited by obstructions and not by the choice of the individual drivers. On the parkways quite the reverse is true, since no obstructions are present, except the speed limits of 40 miles per hour on the Hutchinson River Parkway and 45 (since raised to 50) miles per hour on the Merritt Parkway.

Gasoline consumption on the Post Road amounted to 0.0543 gallons per mile or 18.4 miles per gallon. This rate of consumption is equivalent to a cost of 0.90 cents per car mile. The low rates of speed prevailing on the Post Road required that the car be operated much of the time in the most efficient speed range for gasoline consumption. The saving in gasoline from this cause compensated to some extent for the additional amount used in traffic stops, so that the average gasoline mileage was not particularly low.

The results obtained for gasoline consumption and travel time on the Post Road were quite consistent; the maximum variation of an individual run from the average was about 6 per cent. Most of the results came within 1 or 2 per cent of the average regardless of the day of week or time of day. The traffic was heavier than normal on Sundays and on the July 4th holiday, but no unusual congestion resulted. This was partly due to the relief offered by the parkways and also due to the absence of heavy commercial traffic and to the special efforts of the police of the several cities and towns to keep through traffic moving. Before the parkways were built traffic jams of many hours' duration often occurred on holiday week-ends.

The runs on the parkways were free from interruptions except for a momentary stop at the toll house. On some runs the fare was delivered to the toll keeper without actually bringing the car to a complete stop. The car was driven at the average pace of traffic using the parkways, which resulted in an average speed of 35.7 miles per hour on the Hutchinson River Parkway and 39.6 miles per hour on the Merritt Parkway. The average speed on the Merritt Parkway was substantially lower than the posted speed limit of 45 miles per hour. Evidently traffic as a whole did not take full advantage of the maximum speed allowed.

The average rate of gasoline consumption on the parkways was 0.0497 gallons per mile or 20.1 miles to the gallon, a decrease in gasoline consumption of 8.5 per cent over that required on the Post Road. There were several important factors influencing the rate of gasoline consumption on the parkways which tended to prevent greater savings. These were

TABLE 5

SUMMARY OF RESULTS OF TEST RUNS ON THE BOSTON POST ROAD, HUTCHINSON RIVER PARKWAY, MERRITT PARKWAY AND CONNECTING ROUTES

Route	Distance, miles	Interruptions			Time studies			Gasoline consumption			Gasoline cost*	
		Number of stops	Stops per mile	Seconds per stop	Elapsed time in minutes	Percent of time stopped	Average miles per hour	Gallons used	Gallons per mile	Miles per gallons	Total cost	Cost per mile
											(*Cents*)	(*Cents*)
BOSTON POST ROAD												
Pelham Manor to Port Chester	11.84	11.3	.954	25.4	30.0	15.7	23.7	0.639	.0539	18.5	10.6	.89
Port Chester to Westport	20.96	18.3	.873	22.8	50.1	13.6	25.1	1.154	.0551	18.2	19.0	.91
Westport to Stratford	15.13	11.6	.768	20.1	34.7	11.0	26.2	0.810	.0536	18.7	13.4	.88
Total—Pelham Manor to Stratford	47.93	41.2	.861	22.8	114.8	13.6	25.0	2.603	.0543	18.4	43.0	.90
HUTCHINSON RIVER PARKWAY												
Pelham Manor to Connecticut Line	14.74	0	0	0	24.7	0	35.7	0.730	.0495	20.2	12.0	.81
MERRITT PARKWAY												
New York Line to Westport	21.17	0 5	....	3.5	32.1	0.2	39.6	1.052	.0497	20.1	17.4	.82
Total—Both Parkways	35.91	0.5	....	1.5	56.8	0.1	37.9	1.782	.0497	20.1	29.4	.82
CONNECTING ROUTES												
Route 120A—Port Chester to Parkways	2.50	1.5	.600	14.0	5.2	5.7	29.2	0.130	.0518	19.3	2 1	.85
Route 33—Westport to Parkway	1.97	0.5	.254	8.0	4.6	3 7	25.7	0.104	.0528	18.9	1.7	.86
Route 57—Westport to Parkway	1.69	0.5	.296	15.6	3.9	5.2	26 2	0.088	.0521	19.2	1.4	.86

* Computed at 16.5 cents per gallon.

grades, wind and speed. The parkways traverse a series of ridges and are constantly rising and falling, whereas the Post Road follows the shore line and is more nearly level. The parkways are out in the open, so that cars are exposed to the full pressure of the wind. During the days when tests were made, a breeze of 5 to 10 miles per hour velocity was blowing from the south and southwest. On this account the gasoline required on westbound runs was consistently higher that that required on eastbound runs. The higher speeds prevailing on the parkways required more gasoline per mile; this is a characteristic of nearly all automobiles.

The results show a saving of 0.08 cent per mile in gasoline cost on the parkways compared with the Post Road. If the test car had been driven faster, this differential would have been reduced. The performance characteristics of this car indicate that at 52 miles per hour the gasoline cost per mile on the parkways would be the same as on the Post Road at 25 miles per hour with stop and go traffic. At 52 miles per hour, however, the savings in time by use of the parkways would be nearly 40 per cent greater than given in this report. Such a saving could not be realized, however, without exceeding the legal speed limit.

The results of tests on connecting routes fall about midway between those for the Post Road and the parkways. These connecting roads were much less traveled than the Post Road, yet they were not as unobstructed as the parkways.

Comparison of Results for Boston Post Road and Parkways

A comparison of gasoline cost and travel time for the Post Road and the parkways between Pelham Manor and Westport is shown in Figure 8. In 1939 the Merritt Parkway was open only to Route 33 in Westport, and parkway traffic was obliged to rejoin the Post Road via this route. This arrangement placed the parkways at a disadvantage in making comparisons because the parkway route was about 5 miles longer than the Post Road. Even with this handicap of additional distance, the parkway route cost only 1.5 cents more for gasoline and saved nearly 20 minutes of time.

A more significant comparison will be possible between the parkways and the Post Road after the parkway system has been completed to a junction with the Post Road in Milford (see Fig. 6). Such a comparison has been made in the nature of a forecast in Table 6. In compiling this table the assumption is made that the uncompleted section of the Merritt Parkway will have the same gasoline consumption and travel time characteristics as the sections actually tested. The results for the Post Road have been projected from Stratford to Milford on the same basis.

When the parkway is completed, traffic will be attracted to it and the volume of traffic on the Post Road between Westport and Milford will be reduced. It is doubtful, however, whether the traffic remaining on the Post Road will be greatly benefited, since there will still be as many signal lights, and the same volume of heavy commercial traffic. The district is so thickly settled that the Post Road will always be a heavily traveled road serving a large volume of local traffic as well as all through commercial traffic.

The results of Table 6 indicate that after the Merritt Parkway is completed, the parkway system from Pelham Manor to Milford will be about 2 miles longer than the Post Road but will require about 40 minutes less time and 2.6 cents less gasoline cost. If, however, a driver chooses to drive faster on the parkways than the average speed of the test car, he

TABLE 6

ESTIMATE OF SAVINGS IN TIME AND GASOLINE COST FOR VEHICLES USING PARKWAYS IN PREFERENCE TO BOSTON POST ROAD AFTER THE MERRITT PARKWAY IS COMPLETED

Route	Distance, miles	Elapsed time in minutes	Average speed, miles per hour	Gasoline consumption			Gasoline cost*	
				Gallons used	Gallons per mile	Miles per gallon	Total cost	Cost per mile
							(*Cents*)	(*Cents*)
BOSTON POST ROAD								
Pelham Manor to Stratford (Average of Tests from Table V)	47 93	114.8	25 0	2 603	.0543	18.4	43.0	0.90
Stratford to Proposed Parkway Connection in Milford (Estimated)	5.55	13 3	25.0	0.301	.0543	18.4	5.0	0.90
Total—Post Road from Parkway Crossing in Pelham Manor to Proposed Parkway Connection in Milford	53.48	128.1	25.0	2.904	.0543	18.4	48.0	0.90
PARKWAYS								
Hutchinson (Table V)	14.74	24.7	35.7	0.730	.0495	20.2	12.0	0.81
Merritt (Table V)	21.17	32.1	39.6	1.052	.0497	20.1	17.4	0.82
Merritt under Construction to Route 113 (Estimated)	12 70	19.3	39.6	0.632	.0497	20.1	10.4	0.82
Proposed Extension of Merritt to Connect with Post Road in Milford (Estimated)	6.90	11.6	35.7	0.341	.0495	20.2	5.6	0 81
Total—Parkways from Post Road Crossing in Pelham Manor to Proposed Connection in Milford	55.51	87.7	38.0	2.755	.0497	20.1	45.4	0.82
ESTIMATED SAVINGS Effected by Using Parkways instead of Post Road when Merritt Parkway is completed	2.03 (Increase)	40.4	....	0.149	.0046	....	2.6	0.08

* Computed at 16.5 cents per gallon.

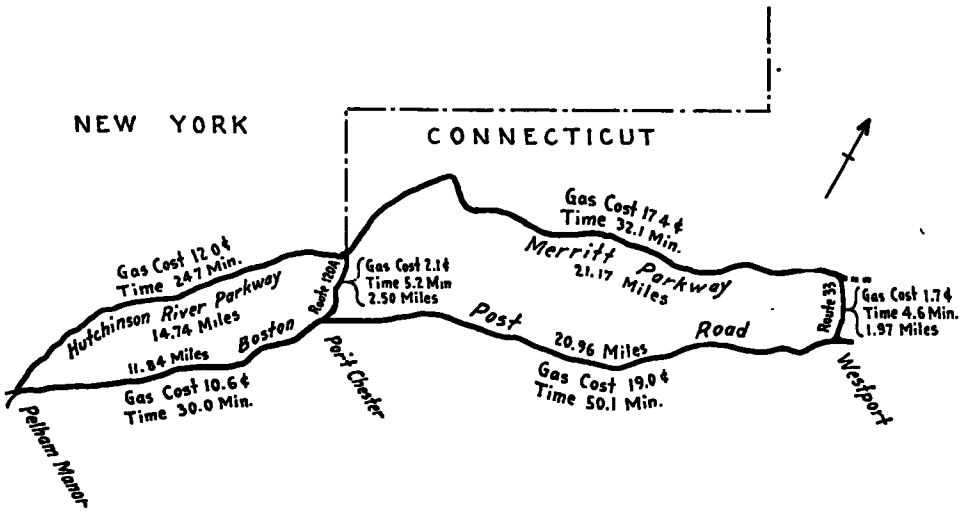


Figure 8. Gasoline Cost and Travel Time—Boston Post Road, Hutchinson River Parkway and Merritt Parkway between Pelham Manor, New York, and Westport, Connecticut, July 1939.

Totals—Pelham Manor to Westport

	Miles	Gas Cost	Time
Via Boston Post Road.....	32.80	29.6¢	80.1 Min.
Via Parkways & Rt. 33.....	37.88	31.1¢	61.4 Min.

Parkway Route is 5.24 miles longer, costs 1.5¢ more for gasoline and saves 18.7 minutes of time.

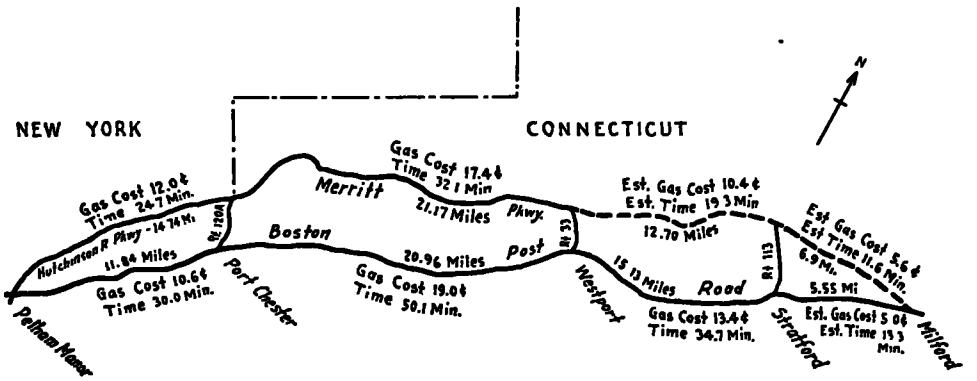


Figure 9. Estimated Gasoline Cost and Travel Time—Boston Post Road compared with Hutchinson River and Merritt Parkways when the latter is extended to Milford, Connecticut.

Totals—Pelham Manor to Milford

	Miles	Gas Cost	Time
Via Boston Post Road.....	53.48	48.0¢	128.1 Min.
Via Parkways	55.51	45.4¢	87.7 Min.

Parkway Route will be about 2 miles longer than Post Road, cost 2.6¢ less for gasoline, and save 40.4 minutes of time.

will save more time and save less in gasoline cost. The comparison is shown graphically in Figure 9.

Other Elements of Vehicle Operating Cost Effected by Traffic Delays.

This report has dealt with but one item of motor vehicle operating cost, that for gasoline. Other items of cost are also effected by traffic delays. Frequent stopping and starting causes excessive tire wear as well as wear on brakes and mechanical parts. If these increments of added expense caused by delays could be evaluated and added to the additional gasoline cost, a very substantial difference would be found between operating costs with and without delays.

The savings in vehicle operating costs which result from highway improvements which eliminate delays are of considerable magnitude and constitute an important factor in the economic justification of such improvements.

DEPARTMENT OF DESIGN

C. N. CONNER, *Chairman*

REPORT OF COMMITTEE ON ANTI-SKID PROPERTIES OF ROAD SURFACES

GEORGE E. MARTIN, *Chairman*

SYNOPSIS

The skid resistance of 36 representative Oregon pavement surfaces was determined by means of a specially constructed trailer towed behind a light truck. A dynamometer in the towing bar measured the tractive resistance of the trailer with one wheel locked, while the trailer was being towed at one of the several constant speeds used in this investigation. Four general types, portland cement concrete, asphaltic concrete, bituminous macadam, and oil mat, were tested. Ages of pavements ranged from 18 years to those not yet opened to general traffic. Surface texture varied from overly fat asphaltic concrete to new heavily grooved portland cement concrete and coarse open textured asphaltic concrete and bituminous macadam.

Tests of the length of skid with wheels locked of a sedan on eleven types of pavement surface in Cleveland, Ohio, are also reported. The standard hydraulic brakes on the car were arranged for air operation, and the length of skid was determined by pistol shot marks on the pavement.

For several years the Committee has advocated the routine use of machines for testing the skid resistance of pavements by state highway departments, municipalities, and other organizations in charge of highways. Recently the State of Oregon ran a number of tests using a machine similar to that described by Professor Moyer[1] on typical pavement sections in Oregon.[2] The Committee presents herewith a description of the methods used in Oregon and the extent of the experimental work.

OREGON SKIDDING TESTS

This is a preliminary report on the skid resistant characteristics of a group of 36 representative pavement surfaces on the Oregon highway system. The pavement surfaces tested include four different general pavement types: portland cement concrete, asphaltic concrete, bituminous macadam, and oil mat surfaces.

The pavements range in age from those laid in 1920 to new pavements laid in 1938 and tested before being subjected to highway traffic. A wide variety of surface textures is included in the test series, ranging from the polished surfaces of overly fat asphaltic concrete pavements, where bleeding of the asphalt has covered the entire surface with free asphalt, and old portland cement concrete pavements worn by seventeen years of traffic, to the new, heavily grooved portland cement concrete pavements and the coarse and open-textured asphaltic concrete and bituminous macadam surfaces.

The tests were made with a specially constructed trailer towed behind a light truck. A dynamometer was placed in the towing bar to record the tractive resistance of the trailer. The tests were made by locking one wheel of the trailer while a constant speed was maintained by the towing vehicle. The tires used were 5.50 by 17 in. retreaded to give a smooth surface without tread design.

Tests were run at constant speeds of 10, 20, 30, and 40 miles per hour on level

[1] *Proceedings*, Highway Research Board, Vol. 13, p. 123. (1933).

[2] Technical Report 39-5, Oregon State Highway Department.

sections of highway under both wet and dry conditions. The results shown are the average of at least four tests. Test data in all cases are reported as coefficients of friction. For tests on surfaces in the wet condition, a sprinkling tank truck was driven ahead of the test vehicle.

The main series of tests was made with a wheel load of 800 lb. which corresponds closely to the wheel load of the average passenger automobile. To correlate the tests with the work of other investigators and to bring out the effect of varying wheel loads, the tests were repeated on a few surfaces using a wheel load of 400 lb.

Three of the pavement surfaces on which tests had been made near the end of the summer season when dirt and oil from traffic had accumulated were retested after a period of several weeks intermittent rain. These few tests indicate the effect of surface cleanliness.

In general the tests confirm the actual driving experience that any of the pavements are satisfactory at reasonable speeds and under dry conditions. At high speeds and with wet surfaces a wide variation in skid resistance was observed. The smooth and so-called sandy-textured surfaces having coefficients of friction when dry of between 0.40 and 0.60 drop to values in the range of 0.20 to 0.30 and in a few cases to below 0.20. The coarser-textured surfaces such as the heavily broomed portland cement concrete, the open type asphaltic concrete, and the newer type bituminous macadam are only slightly affected by water. It was found that the surface textures which permit the draining away of water without flooding the surface are more skid resistant when wet than when dry.

The decrease in skid resistance with increasing wheel load was found to be an important factor especially on dry surfaces. Doubling the wheel load resulted in a decrease in the coefficient of friction of from 25 to 40 per cent on dry surfaces and from 15 to 25 per cent on wet surfaces.

CLEVELAND TESTS

The following description of the work done in the City of Cleveland was presented by Mr. K. H. Rudd of the Standard Oil Company of Ohio.

A 1938 Buick Century four-door sedan was equipped so as to air operate the standard set of hydraulic brakes. The equipment used was manufactured by the Midland Steel Products Company, Cleveland, and was installed by the L. O. Halverstadt Company according to Midland engineers' recommendations.

The set-up is as follows: A 4¼ in. by 3½ in. main air cylinder (part N3050) was rigidly mounted in a steel framework in alignment with the 1⅛ in. diameter hydraulic cylinder supplied with the car. When this was done, the oil cylinder could be actuated to the full extent of its travel by the air cylinder instead of by the foot brake pedal. To control the quantity or pressure of air admitted to the main air cylinder, two valves were used. One was mounted so as to be operated by the brake pedal and the other by a hand lever directly under the steering wheel. The foot valve (part N2111) was of the full compensating type which permitted gradual applications and releases. The hand valve (part N2185) was of the quick opening type and was operated by a high left cam. Either hand or foot valve could be operated independently of the other. The total braking effort was not changed in any way as the original shoes and hydraulic cylinders are still being used in all four wheels.

As the compressor and main air storage system are used for other purposes at pressure of 135 lb., it was considered advisable to use a pressure regulator in connection with the brake operation so as not to cause excessive pressures on

flexible tubing, fittings, wheel cylinders, etc. This made it possible to regulate the maximum pressure to the main air cylinder by fully opening either hand or foot valve and thereby the maximum braking effort could be regulated at will. It was found that a 50 lb. air cylinder pressure was sufficient to lock all four wheels when traveling at ordinary test speeds. The above installation permitted the operation of the braking system in a uniform manner on every test entirely eliminating the human element.

A Projec-tell Meter, purchased from Triangle Associates, 1033 Cathedral Street, Baltimore, Maryland, was also installed on the Buick Sedan. This apparatus automatically and accurately marks the pavement the instant the operator begins the braking process.

This pavement marking device consists of a properly housed revolver loaded with paint pigment cartridges which is fired by an electrically energized mechanism actuated by a contact switch attached to the braking system.

The revolver is fired and an accurate measurement is made from this mark on the road surface to the point (in the same line with the normal progress of the motor car) directly underneath the revolver at the end of the skid.

To similate conditions which the average driver meets, tires which had been driven approximately 8,000 miles, with the tread completely worn, were used. Streets selected for examination were flushed thoroughly by a regular sprinkling truck just before each series of three tests. The sprinkling was extended over a sufficient area to allow the tires of the car to become thoroughly wet before the brakes were applied.

The results are given in Table 1.

TABLE 1

CLEVELAND SKID TEST DATA

Tests Run November 1, 1939

(On Wet Pavements)

Location	Type pavement	Average skid			
		20 m.p.h.		30 m.p.h.	
		Ft.	*In.*	*Ft.*	*In.*
Lakeshore Drive, E. of 53rd	Asphaltic concrete, Ohio T-35.	17	0	36	7
Lakeshore Drive, Btwn. 49-53rd	Asphalt penetration macadam.	17	3	38	4
Buckley Blvd., W. 38	Roughened portland cement, concrete	18	0	48	7
Lakeshore Drive, Btwn. E40-49	New sheet asphalt	19	8	49	11
New Maine Ave., High Level Bridge.	Six-inch steel squares filled with cement concrete	20	4	46	8
Wallworth Run, E. of Fulton	New brick filled with cement grout	20	5	46	0
Madison Ave., W. 71st St.	New brick-asphalt filler— Excess filler removed	21	5	50	3
Buckley Blvd., W. 38	Portland cement concrete treated AE—surface covered with asphalt emulsion for curing purposes	24	3	62	0
W. 51st Train and Clark	Old brick-cement grout filler	27	2	65	2
Superior Ave., E. of 26th	Red medina block-cement grout filler	34	3	90	0
Wallworth Run, W. of Fulton	Old brick-asphalt filler—Excess of asphalt on surface	46	0	108	8

REPORT OF COMMITTEE ON GUARD RAIL

GEORGE A. RAHN, *Chairman*

SYNOPSIS

The Committee report reviews the development of guard rails from the early periods when stone or wood was the material, to the present when a large number of types are in use. It points out that the tensile strength of guard rail cable has been tripled since 1919. Standardization of guard rail insofar as interchangeability of parts is concerned is recommended. Such standardization of parts would lead to simpler maintenance by reducing the number of replacement parts carried in stock and admit of more competitive bidding.

In the development of modern highways and the attendant evolution of vehicle speed, it became apparent to highway engineers that in addition to location, design and construction, another feature called for serious consideration, safety.

In 1928, at the Eighth Annual Meeting of the Highway Research Board, H. S. Mattimore presented a "Report on Guard Fence Research."[1] This report covered the evolution of the guard rail, through the medium of field tests, from the wooden barrier up to and including the two cable rigid offset design.

Since 1928 we have seen the further evolution of the cable rail and the introduction into the field of steel plate, woven wire mesh, steel beam, wire tape, steel strip and others. In attempting to evaluate these some highway engineers resorted to overall tensile strength tests in the laboratory, others to field tests, or a combination of both. Practical experience has taught us that the field test is the desirable one.

In 1933 the State Highway Department of Georgia ran a series of field and laboratory tests under the direction of Mr. S. B. Slack. Again the same year the Pittsburgh Testing Laboratory, under the direction of Mr. P. J. Freeman, made a series of tests for the Toncan Culvert Manufacturer's Association. In 1934 the State Highway Department of Missouri ran a series of field tests under the direction of Mr. F. V. Reagel. In 1934 the Texas State Highway Department ran a series of field tests under the supervision of Mr. J. B. Early. As the result of these tests some designs were found acceptable and others were not.

Changes have since been made in some of these designs, which brings up the problem of evaluation as of the present writing. Confusion exists in the minds of engineers relative to design and efficiency as witness the statement of Folwell and Hardenbergh, who editorialized in the February 1938 issue of Public Works as follows:

"WHAT ARE HIGHWAY GUARD RAILS FOR?"

"Our perhaps elementary understanding of highway guard rails has always led us to the conclusion that they were safety factors, designed to protect the traveling public from hazards at certain danger points along highways. Unfortunately, much competition has developed among various manufacturers, and this competition seems to have reached the point where some engineers are either confused or are bewildered by patents and patent suits, or threats of them. In some cases specifications have gotten tangled up, too. Personally we don't know much about writing specifications for guard rails and in the present state of affairs we don't crave the job of writing them; nor of buying guard rails either. Isn't it about time that one of our stronger technical associations appointed a top-notch committee to so straighten out these matters that the buyers of guard rails can give first consideration to the matter which is of first importance—the safety of the traveling public?"

[1] *Proceedings*, Highway Research Board, Vol. 8.

It is not the purpose of this Committee to expound on the merits or demerits of type or design. What we are primarily interested in is—will the structure do the job? In other words; has it the strength and durability, necessary to protect modern traffic.

We have mentioned the evolution of the guard rail, and also the reason for this evolution. Upon graduating from the horse and buggy days the horseless buggy jumped the approximate travel rate from *five* to the "damning" speed of *fifteen* miles per hour. Now the same horseless carriage is traveling at legal speeds of from *forty* to *fifty-five* miles per hour, and, in some places speed limits have been eliminated entirely.

Now let us see what highway engineers have been doing with guard rails during this time. Prior to 1919 guard rails were built of wood or stone, the former predominating, with some cable. From 1919 to 1924 the combined tensile strength on an average cable guard ranged between 20,000 and 30,000 lb. From 1924 to 1932—the combined tensile strength of the guard ranged between 40,000 and 50,000 lb. From 1932 to the present, the strengths of the various guards have been increased and now average between 70,000 and 90,000 lb. in tension. In this short summary you can see that the engineer is endeavoring to keep up with the safety features demanded by modern traffic. With the increase in traffic speeds the problem of suitable guard rail becomes more serious and complex. Up to the present it has been a case of meeting a price; from now on, especially on our express highways, the problem becomes that of designing to meet conditions. Dealing with the pleasure car alone is not sufficient. Bus traffic, carrying thousands of passengers during the course of the year must be protected. Trucks,—what about them? We recognize them as part of our ever growing traffic volume. Is not some thought necessary in this direction?

In conjunction with considering numerous phases of the modern highway guard rail, thought must be given to standardization, insofar as the interchangeability of parts is concerned. The Problem Committee on Guard Rail, of the American Road Builders' Association, has tackled this problem and is showing some results. Standardization is the only means of placing guard rail in line for competitive bidding, and of cutting down the number of parts which must be carried by a maintenance organization in order properly to maintain its guard rail in a safe condition at all times; also for providing against the eventuality of a manufacturer discontinuing production, which has happened and is happening today.

It is the opinion of the committee that the problem, while complex, is not incapable of solution and its aims are in that direction.

DESIGN LOADS FOR GUARDRAILS

By Joseph Barnett

Senior Highway Design Engineer, Public Roads Administration

SYNOPSIS

The load on a guardrail consists of a moving vehicle approaching the rail at some angle and contacting the rail at the general level of the wheels. The weight of passenger vehicle which should be used in design was obtained from new car registrations. The weights of trucks and busses were obtained from highway planning survey data of a representative group of States. Speeds which should be used in design were obtained from traffic studies made in Illinois and Maryland corroborated by reports of others. Height of contact was determined from a study of wheel sizes. Angle of contact was assumed based on possible skidding around curves.

It is recommended that guardrail be constructed with the center 19 inches above ground to resist vehicles approaching it at an angle of 16 degrees. If passenger vehicles only are to be considered the guardrail should be designed to resist a vehicle weighing 4,000 lb. moving at 50 miles per hour for roads whose design speeds are 60 and 70 miles per hour and at 40 miles per hour for roads whose design speeds are 30, 40 and 50 miles per hour. If allowance is to be made for busses a vehicle weighing 20,000 lb. moving at the same speeds as passenger vehicles should be used in design. If allowance is to be made for trucks a vehicle weighing 14,000 lb. moving at 40 and 32 miles per hour, respectively, should be used in design.

The use of guardrail for highways varies considerably. Guardrails generally are constructed at hazardous locations where a vehicle leaving the roadway is likely to be damaged and the occupants injured. The likelihood of serious injury when a vehicle leaves the roadway depends upon details such as height of embankment and surrounding topographic conditions. The Special Committee on Administrative Design Policies of the American Association of State Highway Officials recommends that "Guardrail should be provided at points of extreme danger. Generally such points are considered to be fills over 6 ft. in height at locations where topographic or other conditions, such as sharp change in alinement, force a material reduction in speed. They may be omitted where it is practicable to provide slopes of 1 on 4 or flatter."

The type of guardrail used varies considerably. Those used more or less extensively at the present time are earth mounds, boulders, masonry of stone and other materials, timber, cable, wire mesh in one wide strand or a few narrow strands, steel plates in one band or a few narrow bands and steel plates formed into shapes to provide beam resistance. Rails which do not rest directly on the ground are supported by boulders or posts embedded in the ground. Posts generally are of timber, steel, or concrete.

The design of guardrail has not, so far as the writer knows, been developed on an engineering basis; that is, on a basis comparable to the design of other highway structures, by determining or assuming loads, assuming reasonably accurate behavior of loads and structure and applying engineering science to determine the stresses and strains to which the various parts are subjected. The reasons for our inability to develop a design of this character are not hard to find. The variables are not only many but each one is great in extent and tests on guardrails made thus far have contributed little to narrow them down so that reasonably accurate assumptions for the variables can be made. Tests have generally consisted of rolling a vehicle

without driver into a complete guardrail and noting the behavior of guardrail and vehicle. Little is known regarding the amount of the initial kinetic energy which is converted into heat or "absorbed" by the crumpling of fenders and other parts of the vehicle, by the crumpling of parts of the rail itself, and by the movement of the posts in the ground. Likewise little is known of the amount of kinetic energy which is converted into potential energy and then returned to the vehicle, now moving in a different direction, by the swaying of the vehicle on its springs, by the deflection and partial return of the guardrail and by the compression and partial return of the earth at the posts; nor do we know how much of the energy should be returned to the vehicle in the form of kinetic energy to result in the least injury to the occupants of the vehicle.

A fact which tends to multiply many of the problems of design is that different types of rails act and are intended to act in different ways. A stone wall, for example, subordinates most other purposes of guardrail to prevention of the vehicle from leaving the road and pays little regard to damage to vehicle or injury to occupants. Some resilient rails may fail when contacted by heavy vehicles at high speed but deflect light vehicles at relatively low speed with little injury to occupants and damage to vehicle and with as little damage to the rail as possible.

This paper does not attempt to answer many of the above questions on guardrail design, many of which will not be answered until proper tests are developed and made, but is confined to the one problem common to all types of guardrail, that of the loads which should be used in design.

The essential features of guardrail design loads may be considered to be the following:

(a) Weight of vehicle.
(b) Speed of vehicle.
(c) Height of contact.
(d) Height of center of gravity of vehicle.
(e) Angle of contact.

Once contact is made many actions tend to reduce the load. The dissipation of energy in crumpled fenders and other parts and the storing up and return of energy in the springs of the vehicle have been mentioned. Other factors are frictional resistance between tires and surface of shoulder and the secondary impact.

When a vehicle approaches a guardrail at an acute angle the front wheels frequently are turned by first contact after which the friction between front tires and the ground, generally the shoulder, aid in turning the vehicle away from the guardrail reducing the kinetic energy resisted by the rail. The rear of the vehicle swings around and contacts the rail some time after the first impact. An appreciable percentage of the mass of the vehicle is still moving after initial impact so that the guardrail on initial impact is called upon to resist kinetic energy much less than that represented by the mass of the whole vehicle and its speed. Looking at it another way the entire kinetic energy is resisted by two impacts the initial one and the secondary one occasioned by the rear of the vehicle swinging around. Tests made thus far clearly show this action and indicate that while the secondary impact is appreciable most of the kinetic energy is resisted by the initial impact. No data are available, however, which give the amount or percentage of the kinetic energy which is resisted by each impact.

WEIGHT OF VEHICLE

Two general methods of approach may be used in determining the weight of vehicle which should be used in the design of guardrail. In one the distribution of weights of all vehicles now on the road is used. In the other the distribution of

weights of vehicles produced in the last two or three years is used. The latter method is preferable because it is more likely to be representative of vehicles on the road in future years.

In the case of passenger vehicles distribution of weights by the latter method was obtained. New car registrations for of all passenger vehicles weighing less than the indicated value. Allowances of 150 lb. for water, gas, and oil and 400 lb. for passengers and baggage were made.

The dashed line curve of Figure 1 represents the distribution of weights of passenger vehicles taken from a report of the State-wide Planning Survey of Iowa.

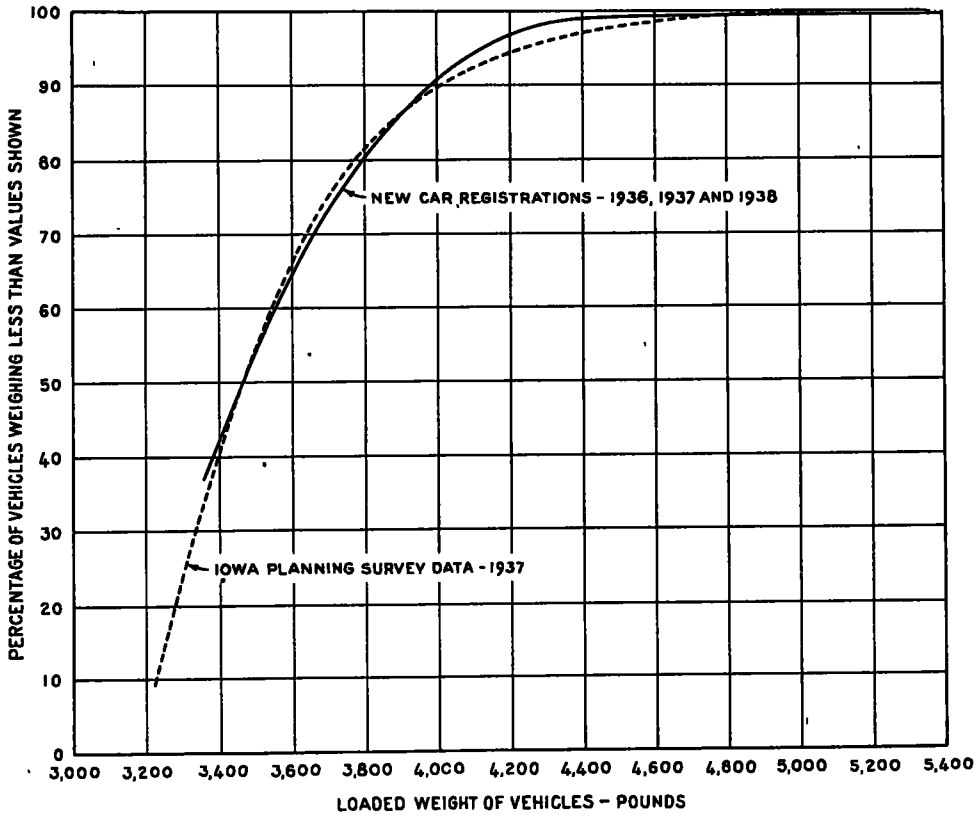


Figure 1. Distribution of Weights of Passenger Vehicles

the years 1936, 1937, and 1938 for the various makes of passenger vehicles were used. These figures were broken down into those for different models and sometimes into those for different body types by the manufacturers who cooperated wholeheartedly. The resulting distribution of weights of passenger vehicles is shown in the solid line curve of Figure 1 any point of which gives the percentage

Vehicles were weighed at three pit-scale stations. All passenger vehicles were weighed regardless of age, condition or load. The vehicles included 1920 to 1937 models.

The weight of vehicle heavier than nearly all passenger vehicles and the weights of vehicles heavier than 90 percent and 67 percent of all passenger vehicles are shown in Table 1.

In the case of trucks and busses the most complete data regarding distribution of weights are those of the highway planning surveys of the several States. New trucks and bus data are inadequate principally because new trucks and busses are sold and delivered in various forms. Sometimes chassis only are sold, in which case little is known regarding the weight of the body and the pay load. In addition users frequently change to larger tires to increase capacity. Sometimes complete trucks are sold but in these cases there is considerable variation in the pay load, which, unlike passenger vehicles, is an appreciable percentage of the total load.

TABLE 1

WEIGHTS OF VEHICLES HEAVIER THAN VARIOUS PERCENTAGES OF ALL VEHICLES

Type of vehicle	Percentage of vehicles accommodated		
	Nearly all	90 percent	67 percent
Passenger vehicles..	4,400	4,000	3,600
Single unit trucks...	19,000	14,000	8,000
Semi-trailers.....	42,000	34,000	25,000
Full trailers.....	75,000	60,000	39,000
Busses.....	28,000	24,000	20,000
All trucks and busses	34,000	21,000	10,500

Weights of trucks and busses were taken from the data of 766 loadometer stations of the highway planning surveys of eleven States. Nearly 800,000 trucks and busses were used in the tabulation. A picture of the sample, its distribution among the States and its distribution among the types of trucks and busses can be had from Tables 2 and 3. Loaded and unloaded trucks were both weighed as they appeared but separate records were kept. Table 4 shows the percentages of trucks and busses which were loaded.

The distribution of the weights of all trucks and busses both loaded and unloaded are shown in the curves of Figure 2 any point of which gives the percentage of all vehicles weighing less than the corresponding value. The weights of vehicles which are heavier than nearly all trucks and busses and the weights of vehicles heavier than 90 percent and 67 percent of all trucks and busses are also shown in Table 1.

The extent to which each type of vehicle is used is shown in Table 5 which gives the percentages of use based on vehicle miles of travel. These data were obtained from the same source used to obtain weight distributions, namely, the highway planning surveys of a representative group of States, but are the results of preliminary tabulations. The small percentage of bus travel is particularly noteworthy.

Little consideration was given to semi-trailer and full trailer combinations in choosing the values shown in the conclusions recommended for designing guardrail for trucks. These combinations are so heavy, for the most part, that the construction of guardrail to resist them would entail expense which may not be justified in view of the limited usage compared to that of single unit trucks. If the rail is a resilient type a design to resist these heavy vehicle combinations would be so stiff that light vehicles would be penalized in that they might, in contacting the rail, just as well hit a brick wall. In general first consideration should be given to the safety of occupants. Semi-trailer and full trailer combinations in addition to relatively little usage rarely have more than two occupants whereas busses generally have ten or more.

Further justification for giving less consideration to semi-trailers and full trailers than to single unit trucks lies in the thought that trailer combinations probably travel at lower speeds than single unit trucks though factual data supporting this thought are lacking. If it be true, considerably greater percent-

TABLE 2

DETAILS OF SAMPLE FOR TRUCK AND BUS WEIGHTS—DISTRIBUTION AMONG THE STATES

State	Number of Loado-meter stations	Single-unit trucks		Semi-trailers		Full trailers		Busses		Total	
		Number weighed	Percent of sample	Number weighed	Percent of sample	Number weighed	Percent of sample	Number weighed	Percent of sample	Number weighed	Percent of sample
Arizona	36	17,047	2.8	2,196	1.8	878	8.0	3,134	8.2	23,255	3 0
Iowa	90	100,644	16.2	18,124	15,1	340	3 1	3,770	9.9	122,878	15.6
Maryland	54	29,528	4.8	6,796	5 7	*	*	2,198	5.7	38,522	4.9
Michigan	126	74,578	12.0	22,448	18.8	5,448	49.2	3,452	9.0	105,926	13.4
Montana	54	22,083	3 6	2,318	1.9	752	6.8	3,368	8 8	28,521	3.6
New Hampshire	30	27,966	4.5	2,298	1.9	*	*	902	2.4	31,166	4.0
Tennessee	72	48,056	7 9	5,321	4.4	*	*	3,707	9.7	58,084	7.4
Texas	124	135,352	21.8	27,275	22.8	424	3 8	6,081	15.9	169,132	21.4
Virginia	54	46,664	7.5	11,412	9.5	*	*	4,528	11.9	62,604	7.9
Washington	54	66,885	10.8	8,785	7.3	2,761	25.0	5,025	13,2	83,456	10.5
Wisconsin	72	50,112	8.1	12,930	10.8	451	4.1	2,013	5.3	65,506	8.3
Total	766	619,915	100 0	119,903	100.0	11,054	100.0	38,178	100 0	789,050	100.0

*Negligible sample—not reported.

TABLE 3

DETAILS OF SAMPLE FOR TRUCK AND BUS WEIGHTS

Distribution Among the Types

State	Percentage of sample:				Total number weighed
	Single-unit trucks	Semi-trailers	Full trailers	Busses	
Arizona	73.3	9.4	3.8	13.5	23,255
Iowa	81.8	14.8	0.3	3.1	122,878
Maryland	76.7	17.6	*	5.7	38,522
Michigan	70 4	21.2	5.1	3.3	105,926
Montana	77.5	8.1	2 6	11.8	28,521
New Hampshire	89.8	7.3	*	2.9	31,166
Tennessee	84 5	9.1	*	6.4	58,084
Texas	80.0	16.1	0.3	3.6	169,132
Virginia	74.6	18.2	*	7.2	62,604
Washington	80.2	10.5	3.3	6.0	83,456
Wisconsin	76.5	19 7	0.7	3.1	65,506
Total No.	619,915	119,903	11,054	38,178	789,050
Percent	78.6	15.2	1.4	4.8	100.0

NOTE: *less than 0.05 per cent.

TABLE 4

PERCENTAGES OF TRUCKS AND BUSSES WHICH WERE LOADED

(Total 789,050 units)

State	Single-unit trucks	Semi-trailers	Full trailers	Busses	Total	
					Loaded	Empty
Arizona	56	76	60	48	57.3	42.7
Iowa	60	79	70	80	63.9	36.1
Maryland	64	74	..	78	66.9	33.1
Michigan	65	70	64	85	66.8	33.2
Montana	47	62	67	49	49.2	50.8
New Hampshire	70	77	..	81	71.1	28.9
Tennessee	61	77	..	88	64.2	35.8
Texas	57	66	59	92	59.7	40.3
Virginia	62	72	..	58	63.6	36.4
Washington	56	65	78	90	59.9	40.1
Wisconsin	69	81	68	89	71.8	28.2
Total	60.7	72.2	67.6	76.9	63 3	36.7

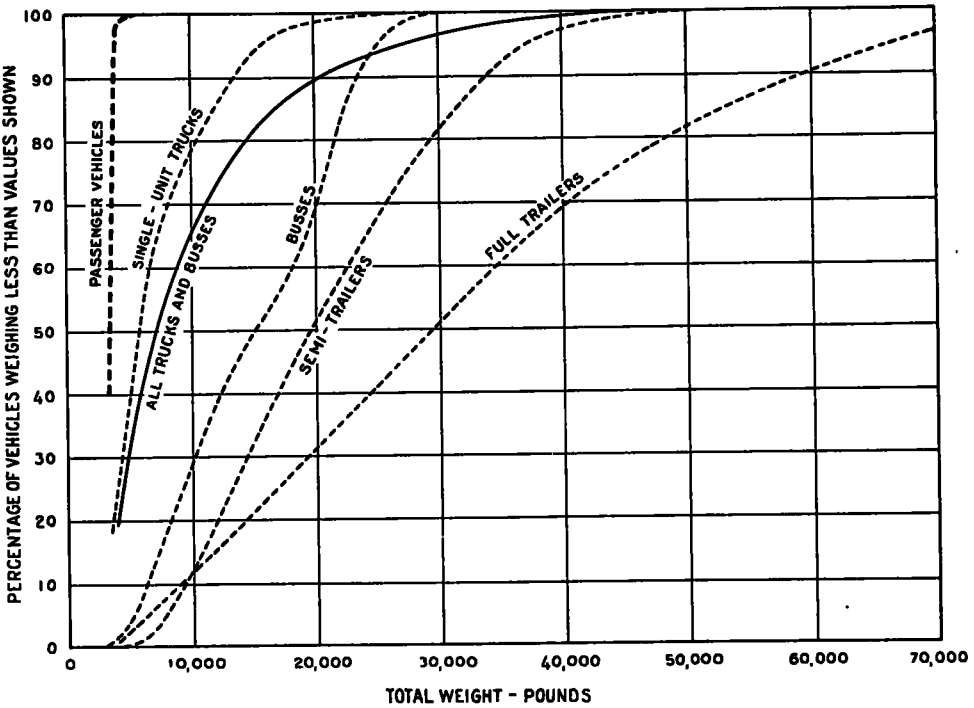


Figure 2. Distribution of Weights of Commercial Vehicles includes Both Loaded and Empty Units

TABLE 5

ROAD USAGE BY TYPES OF VEHICLES

Percentage of annual vehicle-miles on state highway systems, based on preliminary estimates prepared from highway planning survey data; total 23,954,775,000 vehicle-miles

State	Passenger vehicles	Single unit trucks	Semi-trailer and full trailer combinations	Busses
Arizona	81.3	15.4	2.0	1.3
Iowa	81.8	15.0	2.7	0.5
Maryland	80.9	15.2	3.2	0.7
Michigan	84.2	10.6	4.7	0.5
Montana	81.9	15.1	1.8	1.2
New Hampshire	86.1	12.8	1.1	*
Tennessee	78.9	18.6	1.5	1.0
Texas	78.8	16.7	3.7	0.8
Virginia	80.2	15.7	3.0	1.1
Washington	85.2	12.0	1.9	0.9
Wisconsin	86.0	11.0	2.7	0.3
Total	81.78	14.49	3.01	0.72

* Negligible sample.

ages of trailer combinations than those given in the conclusions will be resisted by guardrails designed for the recommended loads.

SPEED OF VEHICLE

The kinetic energy of a moving body may be represented by the equation:

$$KE = \frac{Mv^2}{2g}$$

in which

KE = kinetic energy in foot pounds
M = moving mass in pounds
v = velocity in feet per second
g = acceleration of gravity

Changing to 'V' in miles per hour and substituting 32.2 for 'g' results in the equation:

$$KE = .0335\,Mv^2$$

It is evident that the speed of the vehicle has an important bearing on the design of guardrail, the kinetic energy to be resisted varying as the square of the speed.

In 1937 the Bureau of Public Roads cooperated with the Illinois Division of Highways in conducting a number of capacity studies in which the distribution of the speeds of vehicles was obtained. Charts of various forms are shown in the report[1] of preliminary results. The data are reproduced in different form herein. Speed distribution data also were obtained on a section of road in Maryland on which passing studies were made. The data for these four different road sections are considered. A two-lane tangent in Illinois on which the speeds of 1,741 vehicles were obtained is designated as section A. Section B is a four-lane undivided highway in Illinois on which the speeds of 17,690 vehicles traveling in both directions were obtained. Section C is a four-lane divided highway in Illinois on which the speeds of 9,636 vehicles in one direction were obtained. Section D is a two-lane road practically tangent with a vertical curve on each end. The speeds of 2,517 vehicles were obtained on section D.

The distribution of speeds on each section was obtained for a number of different densities to study the effect of traffic density on speed and interference. Curves showing these effects are included in the report referred to.[1] For purposes of choosing speeds to be used in the design of guardrail no such breakdown is deemed necessary. The speeds of all vehicles measured are included in the distribution used herein. It may be of interest to note, however, that the density for section A, a two-lane road, varied from 168 to 335 vehicles per hour averaging 236 v.p.h., for section B, the four-lane undivided highway, from 518 to 1,869 v.p.h. averaging 1,051 v.p.h., for section C, the four-lane divided highway, from 228 to 1,412 v.p.h. in one direction averaging 1,091 v.p.h., and for section D, a two-lane road, from 374 to 682 v.p.h. averaging 516 v.p.h.

The data are shown in Figure 3 in which a point on a curve represents the percentage of all vehicles traveling less than the indicated speed. Three of the curves represent traffic in Illinois and one in Maryland. The average speeds on the Illinois roads were 41.7, 44.2 and 46.7 miles per hour for the two-lane, four-lane undivided and four-lane divided roads respectively. The average speed on the two-lane road in Maryland was 39.7 miles per hour. Comparison with observations in other sections of the country is desirable.

In 1934 observations[2] on over 2,000 vehicles in northwest Indiana resulted in speed distribution for low speeds nearly the same as curve C in Figure 3 but

[1] Preliminary Results of Highway Capacity Studies by O. K. Normann, *Public Roads*, Vol. 19, No. 12, February 1939.

[2] A Study of Highway Speeds by Prof. Ben H. Petty, Purdue University, *Better Roads*, March 1936.

about 10 or 15 percent higher for the upper range of speeds. The average speed was 50.2 miles per hour.

The speeds of over 22,000 vehicles in California were observed [3] in 1937. For speeds over 45 miles per hour the distribution was about the same as curve B in Figure 3 for the two-lane road in Illinois.

Connecticut observations [4] indicate an average speed for all vehicles of 39.2 miles per hour which is less than the average speeds in Illinois represented by curves A, B and C in Figure 3 and only a trifle less than in Maryland represented by curve D. The same report mentioned the results of other observations showing an average speed of 33.8 miles per hour in Rhode Island in 1935, 35.5 miles per hour in Maryland in 1933, 33 miles per hour in eastern Massachusetts in 1934, 43.7 miles per hour in Michigan, and 43 miles per hour under favorable conditions in Ohio in 1934. From these results the report draws the conclusion that speeds in the east are

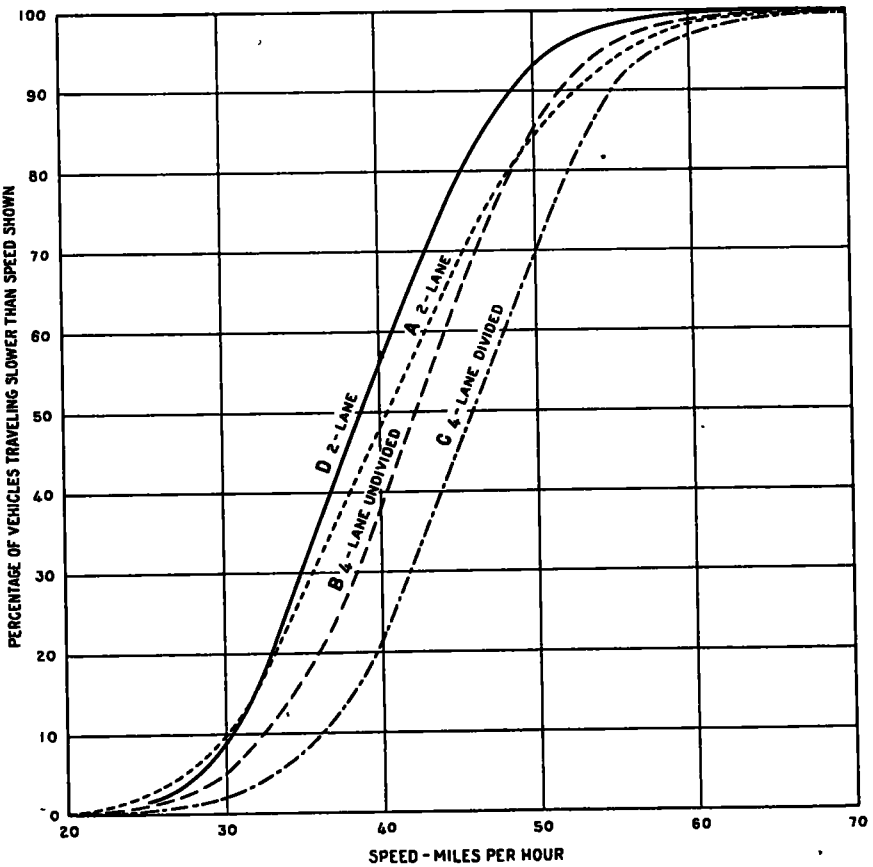


Figure 3. Distribution of Vehicle Speeds on Open Highways with Good Alignment

[3] Annual Statistical Report, 1938, California Department of Motor Vehicles. Abstracted in *Highway Research Abstracts*, June 1938.

[4] Motor Vehicle Speeds on Connecticut Highways by C. J. Tilden, Committee on Transportation, Yale University, 1936. Abstracted in *Highway Research Abstracts*, February 1937.

somewhat lower than those in the west. No data are available for the section of the country between the Mississippi Valley and the Pacific Coastal States but the speeds probably are higher than elsewhere because of the open character of the topography and habitual light traffic densities.

The curves in Figure 3 showing distribution of speeds and the corroborating evidence of other observers cited lead to the following broad conclusions for traffic as a whole on open highways of good alinement, except four-lane divided highways, when density does not impede smooth flow: (The speeds on four-lane divided highways are about 5 miles per hour faster.)

(a) Speeds up to 40 m.p.h. are used by less than half of all drivers.

(b) Nearly half of all vehicles travel between 40 and 50 miles per hour.

(c) A speed of 50 miles per hour is greater than the speeds of about 90 percent of all vehicles.

(d) A speed of 55 miles per hour is greater than the speeds of about 95 percent of all vehicles.

(e) In the near future the fastest 10 percent of all vehicles probably will travel faster than 60 miles per hour.

The curves in Figure 3 and the conclusions drawn therefrom are based on traffic on open roads of good alinement whose design speed is about 60 miles per hour. It is well known that winding alinement and sharp curvature reduce the speed of travel even when traffic is relatively light but comprehensive factual data regarding the extent or pattern of speed distribution on lower speed roads are lacking. The curves in Figure 3 indicate that the speed distribution curve of different speed roads are parallel so that the speed difference is practically a constant for all speeds and not some percentage of the speed. This indication, however, cannot be accepted as conclusive because the data cover only a few roads of good alinement.

In the absence of comprehensive data regarding the effects of highway physical characteristics on the speed of travel it is suggested that the above broad conclusions be deemed to apply to roads whose design speed is about 60 miles per hour and that the speeds cited be multiplied by 60 percent, 75 percent, 90 percent for roads whose design speeds are 30, 40, and 50 miles per hour, respectively. Where the design speed is greater than 60 miles per hour the speeds cited should be multiplied by the ratio the design speed bears to 60. The necessity for even this breakdown is doubtful in developing loads for guardrail design.

Legal speed limits may have some effect upon the speed of vehicles as indicated by various reports of observations made to determine such effects. It is felt, however, that legal speed limits are transitory in character since they inevitably must be adjusted to meet public demand and are observed only to the extent that enforcement is effective. This factor, therefore, should be ignored in determining loads to be used in the design of guardrail.

The traffic whose speed distributions are shown in Figure 3 and the traffic whose average speeds were cited included all vehicles, trucks and busses as well as passenger vehicles. Passenger vehicles and light trucks, up to say 1½ tons capacity which behave like passenger vehicles, constitute such a large percentage of all traffic, rarely less than 80 percent and frequently nearly 100 percent, that the speeds of passenger vehicles alone on roads not impeded by high traffic density are not much greater than those cited. For all practical purposes, therefore, the conclusions regarding speed drawn for traffic as a whole may apply to passenger vehicles.

Trucks travel definitely slower than passenger vehicles, but busses do not travel much slower if at all. The Connecticut observations previously referred to [4] found the average speed of all vehicles to be 39.2 miles per hour, for all passenger vehicles 40.0 an increase of only 2 percent, for trucks 33.8 a decrease of 14 percent, and for busses 41.6 an increase of 6 percent. A report [5] of observations of traffic in Illinois shows that on open rural highways trucks travel at speeds 75 to 80 percent of the speeds of passenger vehicles when the latter travel at 40 to 60 miles per hour. Another report [6] of traffic in Illinois shows that trucks travel at speeds about 70 percent of the speeds of passenger vehicles when conditions are most favorable. Observations [7] of traffic in Rhode Island indicated that the average speed of light trucks was 91 percent, heavy trucks 83 percent and busses 89 percent of the average speed of passenger vehicles.

It may be concluded, therefore, that, under comparable conditions, trucks travel at about 80 percent of the speeds of passenger vehicles. This fact is important in the design of guardrail because the load is in the form of kinetic energy which varies as the square of the speed. The square of the average speed of trucks is only two-thirds of the square of the average speed of passenger vehicles so that a rail designed to resist a chosen weight representing a passenger vehicle can resist a truck weighing 50 percent more.

[5] Report of Special Study on Speed Zoning by D. Grant Mickle, presented at the Safety Congress and Exposition, October 12, 1938.

[6] Operating Speeds and Speed Zoning by H. H. Harrison, Group meeting papers and discussions, American Association of State Highway Officials, annual meeting, 1938.

[7] Some characteristics of Highway Traffic in Rhode Island, *Public Roads*, Vol. 16, No. 11, January 1936.

HEIGHT OF CONTACT

In the days of horse drawn vehicles guardrail closely resembled the ordinary fences used for confining stock in the fields. Early guardrail for motor vehicle traffic was a natural development of fence design and fitted the prevailing high bodied vehicles, large wheels, high center of gravity and slow speed. They generally were constructed to a height of 3 to 4 ft. to contact the vehicles at the general body level.

With the tremendous increase in the speed of travel and the accompanying lowering of the center of gravity and reduction in the size of wheels high guardrails are not only unnecessary but in themselves a source of danger. The most desirable guardrail appears to be one which, in addition to preventing a vehicle from leaving the roadway, will deflect it, preferably to a direction parallel to the road, rather than stop it. The desirable height of a guardrail, therefore, should be such that the chief resisting and deflecting element will be at the general level of the wheels. While the center of gravity of motor vehicles is higher than this level it is not high enough, at least in the case of passenger vehicles, to cause vehicles to overturn. In the guardrail tests observed by the writer there was not one case of a vehicle overturning upon impact and the swaying of the body on contact, caused by the center of gravity being higher than the rail, was clearly discernible. This swaying has the desirable effect of storing some of the kinetic energy into the springs of the vehicle itself, thus cushioning the impact. The danger inherent in a high guardrail is the chance that the swaying body may be snagged by the high portion of the rail impeding its progress and probably resulting in injury to occupants and damage to vehicle.

The trend in the size of wheels of passenger vehicles was found by averag-

ing the sizes of wheels of various makes and models for various years. It is shown in Figure 4. The flattening of the curve appears to indicate that any further reduction in the wheel size of passenger vehicles will not be appreciable. The average diameter of wheels of passenger vehicles manufactured in 1936, 1937 and 1938 is about 29 in. Practically all use tires between 27 and 31 in. in diameter. On this basis the center of the rail element should be about 15 in. above the ground. A reasonable increase in the height of the rail element above ground, however, will result in a rail which will deflect the front wheels of passenger ve-

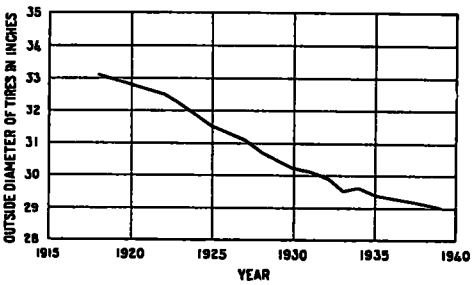


Figure 4. Trend in Outside Diameters of Tires of Passenger Vehicles

hicles just as effectively as a lower one, will be closer to the elevation of the center of gravity and will more closely fit the elevation of bumpers which have been standardized by the Society of Automotive Engineers at 17 in. above ground. In addition a higher rail is more likely effectively to deflect busses and trucks which have decidedly larger wheels than passenger vehicles. Wheel sizes for trucks vary considerably and no reasonably representative value is available but the wheels of busses are more uniform in character. A tire size 9.00 by 20 is used frequently on busses manufactured in the last three years, making the diameter of the outside 38 in.

Low rails at the present time are constructed with the center about 19 in. above ground. There is no indication that it will be necessary to change this practice at the present time.

HEIGHT OF CENTER OF GRAVITY

If the loads for which guardrails are designed were moving solid masses the height of the center of gravity would have a greater bearing on the design of the rail element and the posts than it does. The load is a moving motor vehicle which is deflected when the front wheels contact the rail, whose fenders and other body parts crumple and whose body sways on its springs. A higher center of gravity, particularly if due to a heavier body type, has the effect of increasing the sway of the vehicle on its springs, reducing the impact. An increase in the height of center of gravity does not, therefore, seriously increase the load on a modern guardrail provided, of course, that the height is not great enough to overturn the vehicle or twist the rail element to such an extent as to flatten it out resulting in a serious concentration of stress or mounting of the rail by one or both front wheels.

It is fortunate that the height of center of gravity is not critical because data regarding this value are not available in precise form. The height of the center of gravity of passenger vehicles is about 25 in. and the height is much greater for trucks and busses which is about all we can say about this value at the present time.

ANGLE OF CONTACT

The angle of contact is considered to be the angle between the line of the rail and the path of the vehicle on making contact. It is a very important item in the matter of loads for which a rail should be designed because the modern guardrail is designed to resist very little of the component of the impact parallel to the rail but must resist the component

of the impact perpendicular to the rail. The latter component varies as the sine of the angle of contact which for small angles, almost varies as the angle itself.

Unfortunately few observations have been made regarding the angle at which vehicles actually approach guardrails. If the paths of vehicles were included in the reports of accidents the most probable angle of contact could be determined. When a vehicle gets out of control or is deflected by being hit by another vehicle or by hitting a movable or stationary object the angle of contact may be any angle. A design for an angle of contact of 90 degrees would be required to provide for accidents of this character.

In most accidents on curves when a vehicle approaches at a speed too great for the curve the vehicle skids to the outside and contacts the guardrail at a comparatively small angle. Some measure of the angle of contact may be obtained by assuming that the vehicle proceeds along the tangent of the curve. Larger angles of contact result when larger values are assumed for the distance from the edge of pavement inside which the outer front wheel is located when the vehicle leaves the curve and for the distance from the edge of pavement to face of rail. Reasonably large values are 3 ft. and 6 ft. respectively. The angles of contact for these assumptions are 7 deg. for a curvature of 5 deg. and 16 deg. for a curvature of 25 deg. It is evident that a contact angle of 16 deg. is sufficient to cover nearly all cases of vehicles contacting guardrail in this manner. Most of the guardrail tests made to date have been at an angle of contact of 20 deg.

CONCLUSIONS

Based on the data shown herein it is recommended that guardrail to resist motor vehicle traffic be constructed with the center about 19 in. above ground to resist the following weights and speeds of vehicles approaching the guardrail at an angle of 16 deg.

A guardrail designed for passenger vehicles in accordance with the above will resist 90 percent of all passenger vehicles at the indicated speeds and practically all passenger vehicles at 95 percent of the indicated speeds. It will resist very few busses. Allowing for a reduction of 20 percent in the speed it will resist only 50 percent of single unit trucks and very few semi-trailers and full trailers.

Type of vehicle	Weight in pounds	Speed of vehicle	
		Design speed of road	
		60 and 70 m.p.h.	30, 40 and 50 m.p.h.
Passenger vehicles...	4,000	50	40
Busses.....	20,000	50	40
Trucks.....	14,000	40	32

A guardrail designed for busses in accordance with the above will resist two-thirds of all busses at the indicated speeds and 90 percent of all busses at 91 percent of the indicated speeds. It will resist all passenger vehicles. Allowing for a reduction of 20 percent in the speed it will resist practically all single unit trucks, about 80 percent of semi-trailers and 50 percent of full trailers.

A guardrail designed for trucks in accordance with the above will resist 90 percent of all single unit trucks at the indicated speeds and practically all single unit trucks at 80 percent of the indicated speeds. It will resist only 28 percent of semi-trailers and 20 percent of full trailers. It will resist all passenger vehicles. Allowing for the greater speed of bus travel it will resist only about 25 percent of all busses.

DISCUSSION ON DESIGN LOADS FOR GUARDRAILS

MR. A. E. BRICKMAN, *American Steel and Wire Company:* Having been of the opinion that too many variables enter into the collision of an automobile with a highway guard to permit calculation of design requirements, the American Steel and Wire Company has set up a permanent proving ground duplicating actual conditions, for establishing necessary facts, and proving consistent performance of the resultant products.

Since the proving ground's inception in 1935, some 140 impacts of automobiles with guards have been made. Cars generally of the passenger type, weighing an average of 4,000 lb., collided with the guards at speeds as high as 51 miles per hour. The angle of impact was generally 20 deg. with a tangent guard 150 ft. long. The point of impact may be directly at the post or at any desired position between posts. Summer, with its slack rail elements, as well as mid-winter with its frozen ground and extra taut rail members, are conditions common to highway installations and have been duplicated and tested at this proving ground in Worcester, Massachusetts.

In Test No. 102, conducted December 20, 1938, the following knowns were established in this prescribed manner. Both ends of each cable element in a 150-ft. test section of a standard Pennsylvania three-cable highway guard were connected with devices which permitted, after impact, measurement by laboratory methods of the tension stresses in either direction of each cable element. The totals for the three cables above the initial tension were 69,300 lb. in direction of the course of the car and 25,800 lb. tension in the opposite direction. This knowledge permits us to prepare balanced anchor rod and deadman recommendations in keeping with the requirements and strength of the rest of the guard.

In this Test No. 102 we also established knowns, which permitted us to prepare a mathematical functional analysis of the guard in a cycle of impact. The recapitulation of relatives will allow us better to know the requirements of guards under the common and what we believe to be above average conditions.

Figure 1 shows the Worcester Proving Ground. It has a plank runway 350 ft. long with a 17 percent grade. The test car is driverless, the steering wheel is free, and the speed is established by timing for a covered distance. Angle of impact, post spacing, rail member height, tension, and road edge condition are established for the particular purpose of the test in the erection and relation of the guard to the end of the runway.

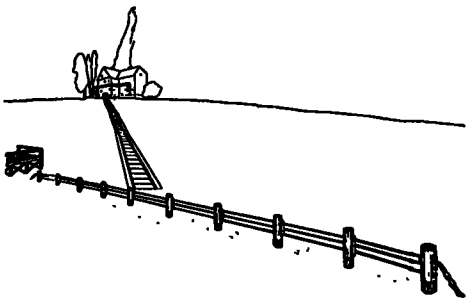


Figure 1. Guardrail Proving Ground

The three cable intermediate post construction which is referred to in the analysis is shown in Figure 2. The four cable construction is similar.

Figure 2 also shows the shape of the bracket, which introduces desirable friction in both compression and recoil.

Figure 3 is referred to as the functional analysis. The center portion of this chart is a plan diagram of Test 102 on which the crosses above the figures indicate known quantities established in this test. The remaining forces, distances and time are the computed resultants.

C^1 is greater than C^2 and shows that the anchorage strength for this severity of impact should exceed 69,300 lb. It also shows that the combined strength of the cable elements should exceed 69,300 lb.

B^2 is greater than B^1 and shows that the resilient off-set post bracket for this type of impact should withstand 15,425 lb. without collapse to provide the desirable resilient function. It further shows that the bracket for this impact must exceed 15,425 lb. in elastic limit to permit salvage comparable to a new bracket.

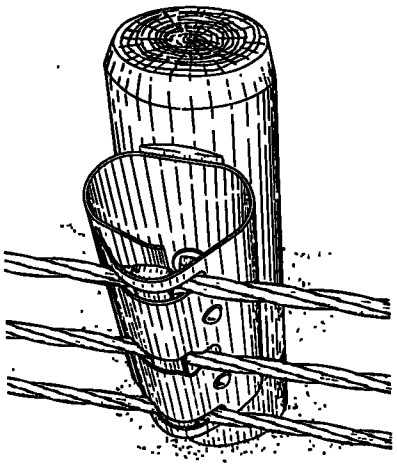


Figure 2. Three-Cable guardrail showing bracket and intermediate post

The upper portion of Figure 3 is devoted to a plan and elevation view which we shall assume as typical. It includes a 10-ft. (right hand) cement concrete travel lane, a 10-ft. rolled gravel shoulder with a highway guard set in 2 ft. from the berm. The distance from the center line of the car to the face of the guard is 13 ft.

With these distances, the car, in order to duplicate the impact by the test car, at an angle of 20 deg., must take the course R^1 which has a radius of 215.5 ft.

The test car weighed 4,300 lb., while the present average passenger car and load we believe is about 3,800 lb.

Referring to C^1, which is the maximum requirement on the cable elements, we find that at 50 m.p.h. the load of 76,483 lb. exceeds the common 75,000-lb. requirement for three cable guard construction. However, when we refer to this same speed under column CF^1, which is the centrifugal force for the 215.5-ft. radius, we find it is 2,551 lb. This greatly exceeds the 60 percent coefficient of friction, which we believe is undisputedly high for tires on gravel. Hence the car would strike the guard at some point beyond at a more oblique angle.

With further reference to this same CF^1 column we find that at 60 m.p.h. or faster the centrifugal force is greater than the weight of the car which would probably result in no guard impact if the 215.5-ft. radius were attempted.

With this in mind we may be led to believe that a guard which would withstand the prescribed Test 102 would also withstand all of the most probable conditions on tangent highways with average car impacts.

If we may assume that 0.60 is a high average for tire coefficient of friction on road shoulders, we may obtain the radius R^2 at which a car of known speed can impact a guard of the shown typical installation, as shown in the right hand table in Figure 3, by the formula:

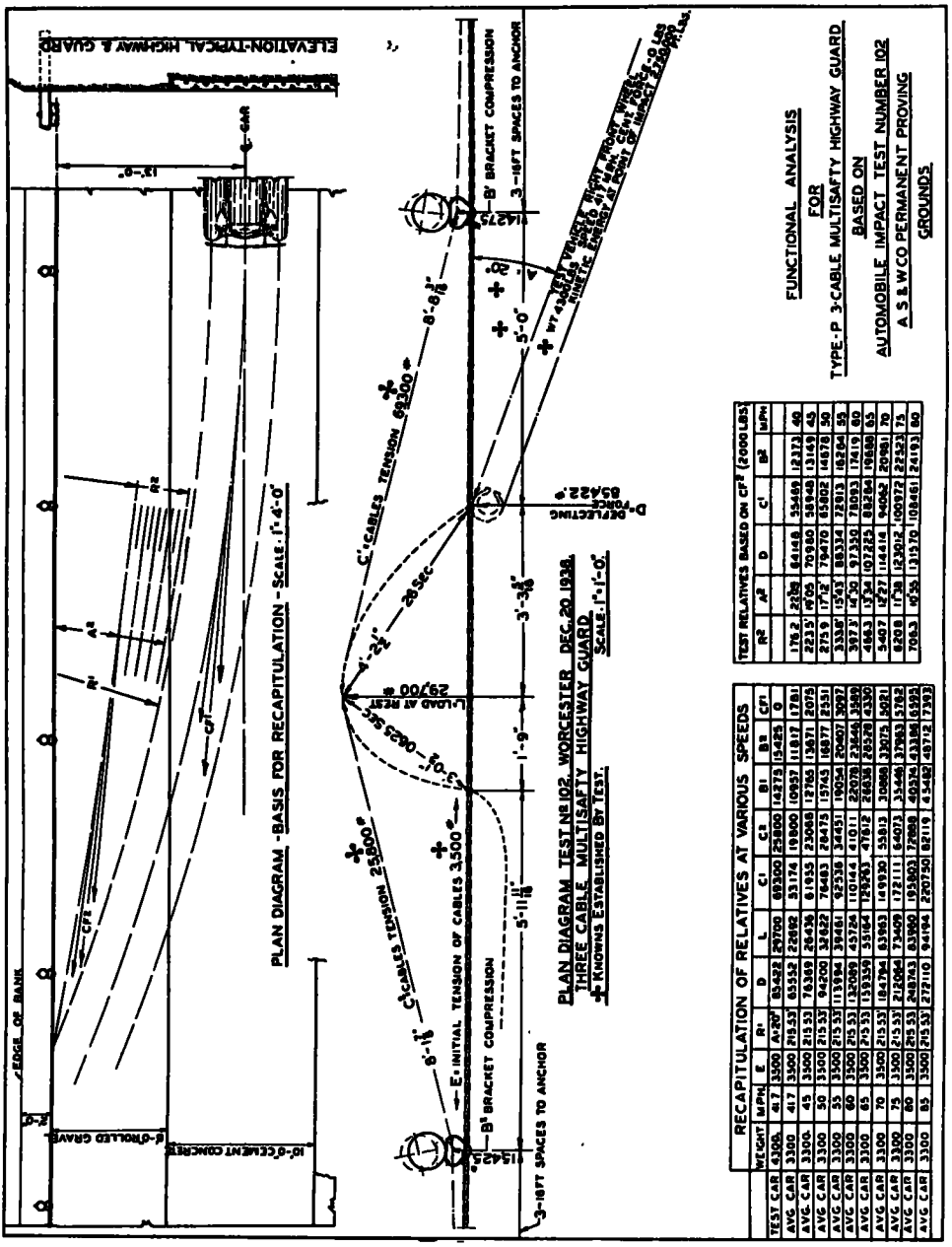
$$R^2 = \frac{W V^2}{F W g}\ ;\ \text{in which}$$

W = Wt. of vehicle
V = Velocity in ft. per sec.
F = Coefficient of friction
g = Acceleration of gravity = 32.16

$$\text{In this case } R^2 = \frac{3{,}300 \times V^2}{2{,}000 \times 32.16}$$

The resultant angle A^2 is also given for each radius R^2 in this table.

With the speeds shown in the right column M.P.H. we find a three-cable



RECAPITULATION OF RELATIVES AT VARIOUS SPEEDS

	WEIGHT	MPH	E	R'	D	L	C'	B'	B''	C#'
TEST CAR	4300	41.7	3500	A=20°	65432	29700	62300	29800	14273	19425
AVG. CAR	3300	41.7	3500	219.33	65532	28982	33174	19800	10937	11817
AVG. CAR	3300	45	3500	219.33	76389	26436	61955	23048	12765	13671
AVG. CAR	3300	50	3500	219.33	94700	34429	76483	28473	15745	16877
AVG. CAR	3300	55	3500	219.33	113994	39461	92336	34491	19054	20407
AVG. CAR	3300	60	3500	219.33	132969	49784	110246	41012	22678	23246
AVG. CAR	3300	65	3500	219.33	153359	57454	128983	47826	26038	28328
AVG. CAR	3300	70	3500	219.33	184794	63963	149930	55813	30888	33075
AVG. CAR	3300	75	3500	219.33	212064	73409	172111	64073	35446	37983
AVG. CAR	3300	80	3500	219.33	240743	83860	195903	72888	40324	43188
AVG. CAR	3300	85	3500	219.33	272110	94184	220750	82119	45482	48712

TEST RELATIVES BASED ON C#' (2000 LBS)

R²	A²	D	C'	B²	MPH
176.2	22.33	64146	35469	12372	
223.5	4.05	70980	38940	13449	45
275.9	17.5	79470	43602	14978	50
335.6	19.43	86334	72813	16704	55
397.3	4.90	97330	78093	17419	60
466.3	17.24	107825	83284	19688	65
540.7	12.27	114414	94062	20981	70
620.8	17.38	123012	100972	22323	75
706.3	4.05	131970	108461	24193	80

Figure 3

75,000-lb. guard will withstand something over 55 m.p.h. and the four cable guard will withstand approximately 75 m.p.h.

These are speeds at the point of impact and we believe it would be safe to assume that most cars had reduced to these speeds from much higher figures since the points of deviation from tangent are 91 and 125 ft. respectively in advance of the point of impact.

Column B^2 gives 22,523 lb. as the compression requirement for the heavier loaded bracket needed to provide a balanced design for the 100,000 lb. tensile strength guard.

These resultants which tend to provide a favorable yet necessary margin of safety for the passenger car are obviously low for trucks, busses, semi- and full trailers.

MR. JOSEPH BARNETT, *Public Roads Administration:* The tests described by Mr. Brickman are of considerable interest because of the measurement of tension in the rail upon vehicle impact; rarely, if ever, measured heretofore. It may be pointed out, however, that the tension measured in this case was that at the end of the test section by means of calibrated springs connecting the rail to the end posts. The tension in the rail itself must have been somewhat greater than the values shown because friction between the rail and the offset bracket at each of the several posts between the point of impact and the end posts undoubtedly decreased the tension in the end springs.

The difference between the tension in the rail ahead of the point of impact and that in the rail behind is surprisingly large indicating considerable friction between vehicle and rail in a longitudinal direction. This is contrary to behavior of vehicles observed on moving pictures of similar tests in which it appeared that the reduction in speed parallel to the rail due to impact was small. It may be that the difference in tension was not as great as that indicated by the end springs. The offset bracket is so constituted that the friction between rail and bracket in a direction parallel to the direction of travel may be smaller than that in the opposite direction.

REPORT OF COMMITTEE ON THE USE OF HIGH ELASTIC STEEL AS REINFORCEMENT FOR CONCRETE

H. J. GILKEY, *Chairman*

FOREWORD

This report constitutes Chapter XI of a continuing project made up of a series of investigations of problems relating to the use of high elastic limit steel as reinforcement for concrete.

The titles and authorship of the past and current chapters are as follows:

Chapter I. Questions and Their Status. Vol. 14 (1934), p. 258-270. (Gilkey and Ernst.)

Chapter II. References and Brief Summaries. Vol. 14 (1934), p. 271-283. (Gilkey and Ernst.)

Chapter III. Design Procedure and Possible Economies from the Use of Higher Design Stresses. Vol. 14 (1934), p. 283-314. (Gilkey and Ernst.)

Chapter IV. Sustained Loading Tests on Slender Concrete Beams Reinforced with High Elastic Limit Steel. Vol. 15 (1935), p. 81-111. (Gilkey and Ernst.)

Chapter V. Pullout Tests for Bond Resistance of High Elastic Limit Steel Bars. (Reconnaissance Series of 1936) Vol. 16 (1936), p. 81-95. (Gilkey and Ernst.)

Chapter VI. An Experimental Study of Bond Stress. Vol. 16 (1936), p. 96-99. (Dunagan and Ernst.)

Chapter VII. Concrete Slabs Reinforced with High Yield Point Steel Bars. Vol. 16 (1936), p. 100-114. (Mylrea.)

Chapter VIII. Bond Resistance of High Elastic Limit Steel Bars; Series of 1937. Vol. 17 (1937) p. 150-186. (Gilkey, Chamberlin and Beal.)

Chapter IX. The Distribution of Strain in the Concrete of Pullout Specimens; Series of 1937, Vol. 18 (1938) p. 114-129. (Gilkey, Chamberlin and Beal.)

Chapter X. The Effects of Impact on Reinforced Concrete Beams. Vol. 18 (1938) p. 130-139. (Mylrea.)

Chapter XI. Bond Tests on Rusted Bars. Vol. 19 (1939) p. 149 (Gilkey, Chamberlin, and Beal.)

The work reported as Chapter XI was conducted at Iowa State College as a cooperative project between the Highway Research Board and the Iowa Engineering Experiment Station.

The results were submitted to the members of the project committee and are recommended by them to the Department of Design for publication. The project committee as now constituted consists of:

G. C. Ernst, Assistant Professor of Civil Engineering, University of Maryland.

T. D. Mylrea, Professor and Head of Civil Engineering, University of Delaware.

F. E. Richart, Research Professor of Theoretical and Applied Mechanics, University of Illinois.

Searcy B. Slack, Consulting Engineer, Decatur, Georgia.

H. J. Gilkey, Professor and Head of Theoretical and Applied Mechanics, Iowa State College, Chairman.

In November 1939 the committee lost by death A. L. Gemeny, Senior Engineer, Public Roads Administration who had been a valued member of the committee since its formation in 1933

CHAPTER XI. BOND TESTS ON RUSTED BARS

By H. J. Gilkey, *Professor and Head*
S. J. Chamberlin, *Assistant Professor*, and R. W. Beal, *Instructor*
Department of Theoretical and Applied Mechanics, Iowa State College

SYNOPSIS

Plain round ⅝-in. rail steel bars were exposed to the weather for 0, 1, 2, 3, 4, 6, 7, and 8 months respectively. Rust formation was observed and measured by abrading the rust and weighing it. On companion bars the rust was: (a) left unmolested, and (b) wiped off with burlap. All bars were vertically cast into 4 by 4 by 10-in. 28-day pullout specimens (L/D = 16) and observations were taken for "first slip," ultimate bond resistance and drag resistance after a slippage of 0.08 in. Amounts of slippage were measured at both the loaded and the unloaded ends of the bars. The general conclusions reached are that the light layer of loose powdery red rust that first forms is of negligible importance and that firm rust tends to increase bond resistance. After the rust became deep, loose and flaky, as for the longest exposures, bond was reduced moderately but wiping the loose rust off with burlap more than restored the bond to that of the unrusted bar. Tests and measurements show that even for the longest exposures which gave deep loose layers of rust there was no significant reduction in bar cross-section because of the rust. The volume of rust is much greater than that of the steel from which it was formed. The results secured are in essential agreement with the limited recorded findings of the other investigators listed as references.

For most climates and most job conditions, it is practically impossible to prevent some rusting of reinforcement prior to its use in concrete. The extent to which rust on bars may effect the bond resistance that they can develop has long been a moot question.

Generally speaking, the practice has been to tolerate a limited amount of firm rust, but the element of personal judgment involved in distinguishing between firm and loose rust and in deciding when the rust was excessive has in itself occasionally been a source of bickering between the materials dealer and the contractor or between the contractor and the inspector or engineer. Even if the classification and measurement of rust were easily accomplished, the fact remains that very little is known about the influence of any specific kind or degree of rust on potential bond resistance.

Such reconnaissance data as have been available (appended references No. 1 to 5) have been reassuring but at best the data are meager.

Accurate quantitative or qualitative measures of rust are important only to the extent that the amount of the rust or its nature is found to have a significant bearing upon the usefulness of the bar, either through its influence upon the effective area of cross-section or through its effect upon the bar surface and the bond that can be developed.

OUTLINE OF THE PROBLEM

Perhaps a series of questions will best set forth some of the interrelated aspects of the general problem.

1. How does each of the following types of rust on bars influence the bond that they can develop as reinforcement: (a) Early-stage rust; a shallow, loose coating that rubs off easily; powdery when dry? (b) Intermediate-stage rust; a firm fairly deep-seated more or less continuous cover only the surface of which can be removed easily by ordinary rubbing? (c) Late-stage rust; thick multiple layers, the outside portions being loose, flaky and easily detached?

2. Such terms as loose and firm; shallow and deep, as applied to rust are indefinite

and are subject to variable interpretations. Is it practicable to devise methods for measuring relative amounts of rust by scraping off and weighing, by successive direct measurements of bar diameters, or by studying the thickness of the rust layers photomicrographically, or otherwise, and to correlate the measured amounts of rust with the bond resistance that can be developed by the bars?

3. Is the progress made by rust formation proportional to the period of exposure in a given environment?

4. For bars of a given diameter, at what stage do the inroads of the rust upon the effective area of cross-section of the bar become significant in reducing its load-carrying capacity?

5. Limited, previous reconnaissances have indicated that firm rust is beneficial rather than detrimental to bond. It seems probable, however, that rust which has reached the stage of multiple layers of loose flaky material must inevitably lower potential bond development. What reasonable measures may there be, if any, to restore at small cost, an over-rusted surface to a condition at least as satisfactory as the norm (the unrusted bar)?

OUTLINE OF THE TESTS

Question 1 called for a graduated series of tests by which rust could be observed and studied in its various states and stages. Preliminary studies showed that the rust itself is not one simple oxide of iron, but that on the same bar there may be present rusts of several different compositions.

For the purpose of this paper rust is considered to be any of the various types of coating that may form on the bar as the result of exposure to moisture. Oxide coatings on some metals are protective in nature in that they form a surface film that prevents penetration of the corrosive influence as long as the film remains intact. Rust is not of this class, since the conditions beneath a layer of rust are favorable to continued rust formation and penetration as long as moisture continues to be available. Once fully embedded in sound concrete rust formation ceases under even rather severe exposure and the problem here considered relates only, therefore, to the rusting of exposed, unembedded bars.

Consideration was given to the desirability of attempting to produce rust rapidly by using salt water or other chemical corrosives, and some preliminary experiments were conducted with this possible procedure in mind, but it was decided that outdoor exposure was preferable, providing the time required to produce measurable different degrees of rust didn't prove to be too great. For one thing, there was the probability that there would be present residually enough of the corrosive agent to react with the cement and thereby complicate the results from the bond tests.

The steel consisted of nine $\frac{5}{8}$-in. diameter plain round rail-steel bars each 10 ft. long. These were part of the same lot of material which was used for the bond tests reported in Chapters VIII and IX (Vol. 17 and 18, *Proceedings* Highway Research Board). Relatively small bars were selected, partially to keep the maximum pull developed by pullout specimens within the capacity of the 20,000 lb. testing machine best adapted for the work, and partially to exaggerate the rust effects because of the greater specific surface of small bars. Plain bars were selected because it was believed that characteristic rust effects could better be identified if the complicating effects of lug action were excluded for the present. Reference No. 5 reports briefly and in part some results from contemporary tests of rust effect on the bond developed by deformed bars.

Figure 1 shows how each bar was sub-

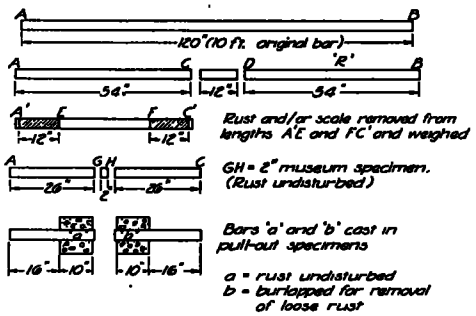


Figure 1. Allocation of Original 10 ft. Length of Bar. Portion marked "R" (DB length) is reserved for possible follow-up tests.

divided and the treatment accorded to each portion. Lengths CD were reserved as unrusted strength controls for the comparative tensile tests reported in Table 1. The relatively short 10-in. embedments (L/D = 16) used for the pullout specimens insured slippage at tensile stresses which were well below the yield point of this steel, as may be noted by comparing the yield points of Table 1 (Col. h) the lowest of which was at a total load of 15,500 lb. (about 50,500 lb. per sq. in.) with the maximum load developed in a pullout test (Table 2 Col. e)

TABLE 1.

COMPARATIVE MEASUREMENTS OF DIAMETERS AND STRENGTHS OF BARS AS CHECKS ON POSSIBLE REDUCTION OF SECTIONS BY RUST

Spec. No.	Length (Fig. 1)	Exposure	Mean diam. (in.)	Max. deviation (spread) (in.)	Av. deviation in diam. (in.)	Apparent reduction in area %	Total load (lb.)		Apparent strength reduction due to rust %	
							Y. P.	Ult.	Y. P.	Ult.
a	b	c	d	e	f	g	h	i	j	k
0 Str. Control..	CD	None	0.627	0.008	.003	...	19,825	32,825	Not	rusted
0 Rust Control.	AG(a)[1]	None	.624	.008	.003	...	19,700	32,875	Not	rusted
0 Rust Control.	HC(b)[1]	None	.625	.009	.002	...	19,925	32,775	Not	rusted
1 Str. Control..	CD	None	0 627	0.008	.003	...	19,825	32,825	Not	rusted
1 Rusted.....	AG(a)	1 Mo.	.624	.010	.003	1.0	19,800	32,600	0.1	0.7
1 Rusted.....	HC(b)	1 Mo.	.625	.009	.003	0.6	19,675	32,650	0.8	0.6
2 Str. Control..	CD	None	0 627	0.007	.002	...	16,520	31,250	Not	rusted
2 Rusted.....	AG(a)	2 Mo.	.626	.009	.003	0.3	16,225	31,125	1.8	0.4
2 Rusted.....	HC(b)	2 Mo.	.627	.007	.002	0.0	16,425	31,275	0.6	0.1[2]
3 Str. Control..	CD	None	0.626	0.008	.002	...	16,600	31,200	Not	rusted
3 Rusted.....	AG(a)	3 Mo.	.625	.007	.002	0.3	16,420	31,175	1.2	0.1
3 Rusted.....	HC(b)	3 Mo.	.624	.008	.002	0 6	16,325.	30,975	1.7	0.7
4 Str. Control..	CD	None	0.629	0.008	.002	...	19,650	32,300	Not	rusted
4 Rusted.....	AG(a)	4 Mo.	.627	.006	.002	0.6	19,150	32,225	2.5	0.2
4 Rusted.....	HC(b)	4 Mo.	.626	.004	.002	1.0	19,450	32,275	1 0	0.1
6 Str. Control .	CD	None	0.628	0.004	.001	...	18,450	30,125	Not	rusted
6 Rusted.....	AG(a)	6 Mo.	.626	.006	.002	0.6	18,300	30,175	0 8	0 2[2]
6 Rusted.....	HC(b)	6 Mo.	.625	.005	.001	1.0	18,320	30,175	0.7	0 2[2]
7 Str. Control..	CD	None	0.630	0.010	.003	...	15,550	25,900	Not	rusted
7 Rusted.....	AG(a)	7 Mo.	.626	.007	.002	1.3	15,725	25,675	1.1	0.9
7 Rusted.....	HC(b)	7 Mo.	.626	.006	.001	1 3	15,500	25,725	0.3	0.7
8 Str. Control .	CD	None	0.628	0.005	.001	...	18,450	30,300	Not	rusted
8 Rusted.....	AG(a)	8 Mo.	.625	.005	.001	1.0	18,375	30,175	0.4	0.4
8 Rusted.....	HC(b)	8 Mo.	.626	.008	.003	0.6	18,350	30,125	0.5	0 6

NOTES: Cols. d, e, and f are from 10 micrometer measurements at approximately one-inch intervals along each bar; adjacent diameters at right angles. [1]Rust Controls No. 0 were from the DB length of Bar No. 1. (See Fig. 1). Strength Controls No. 0 and No. 1 are identical, being CD length of Bar No. 1. [2]Increase.

12,920 lb. (about 42,200 lb. per sq. in.). Most of the yield points were above 60,000 lb. per sq. in. and most of the maximum tensile stresses developed by bond were 40,000 lb. per sq. in. or below. No bar reached or even closely approached its yield point during the pullout test.

The 54-in. DB lengths were stored for possible future tests except that for bar No. 1 which was numbered 0 and used for a pair of unexposed rust controls. The eight AC lengths were exposed, starting July 26, 1938, on an outside west-exposure ledge at the third floor level of the Laboratory of Mechanics, as illustrated in Figure 2. Figure 3 shows the surface texture appearance of the bars just prior to exposure.

At intervals of approximately one

TABLE 2

DATA FROM THE PULLOUT TESTS

Days exposed	Spec. No.	Rust[1] (grams)	Load on bar (lb., above) (lb. per sq. in., below)			Bond stress (p. s. i., above) (% of first slip, below)			Slip at ult. (in.)	
			First slip	Ult.	Drag[2]	First slip	Ult.	Drag[2]	Loaded end	Free end
a	b	c	d	e	f	g	h	i	j	k
0	0	1.004	9530	10,430	7080	474	519	352	0.0125	0.0052
			(31,000)	(34,000)	(23,100)		(109.5)	(74.3)		
	0		10,030	12,330	8180	499	614	407	0.0192	0 0100
			(32,700)	(40,200)	(26,600)		(123.0)	(81.6)		
38	1-a	1.595	10,400	12,370	8480	518	615	422	0 0209	0.0104
			(33,900)	(40,300)	(27,600)		(118.8)	(81.5)		
	1-b		9000	9910	6440	448	493	320	0.0118	0.0053
			(29,300)	(32,300)	(21,000)		(110.0)	(71.5)		
62	2-a	1.676	11,000	12,920	9220	547	643	458	0.0198	0.0098
			(35,900)	(42,200)	(30,100)		(117.5)	(83.7)		
	2-b		9400	11,540	8230	467	575	409	0 0179	0.0101
			(30,600)	(37,600)	(26,800)		(123.1)	(82.5)		
92	3-a	1.366	10,720	11,290	7340	534	561	365	0.0128	0.0048
			(35,000)	(36,800)	(23,900)		(105.0)	(68.5)		
	3-b		8000	10,450	8100	398	520	403	0.0181	0.0100
			(26,100)	(34,100)	(26,400)		(130.7)	(101.1)		
121	4-a	1.811	8000	9480	6130	398	472	305	0.0170	0.0101
			(26,100)	(30,900)	(20,000)		(118.6)	(76.6)		
	4-b		8980	10,600	6980	447	528	347	0.0128	0.0052
			(29,300)	(34,600)	(22,700)		(118.0)	(77.6)		
184	6-a	3.414	9570	10,380	6200	476	516	308	0.0129	0.0050
			(31,200)	(33,800)	(20,200)		(108.3)	(64.6)		
	6-b		9980	11,480	6750	496	571	336	0.0139	0.0050
			(32,500)	(37,400)	(22,000)		(115.0)	(67.8)		
212	7-a	4.371	9420	10,980	6560	468	546	326	0.0175	0.0100
			(30,700)	(35,800)	(21,400)		(116.7)	(69.7)		
	7-b		11,230	12,760	8120	559	634	404	0.0147	0 0050
			(36,700)	(41,600)	(26,500)		(113.3)	(72.3)		
242	8-a	4.597	·8970	9640	5360	446	479	267	0.0112	0.0050
			(29,300)	(31,400)	(17,500)		(107.5)	(59.9)		
	8-b		10,020	12,520	7860	499	624	391	0.0202	0.0100
			(32,700)	(40,800)	(25,600)		(125.0)	(78.4)		

[1]Rust in grams from 24 inches of bar.

[2]Drag is taken as the load (or bond stress) after a slippage of 0.08 in.

Bar 1-a, 2-a, etc., rust undisturbed.

Bar 1-b, 2-b, etc., wiped with burlap.

month, one bar was removed and stored indoors where the air was relatively dry and presumably no further rusting or surface change occurred. The last bar, damaged through misuse by a laboratory assistant and there is, therefore, one gap in the series.

Figure 4 is a close-up photograph of

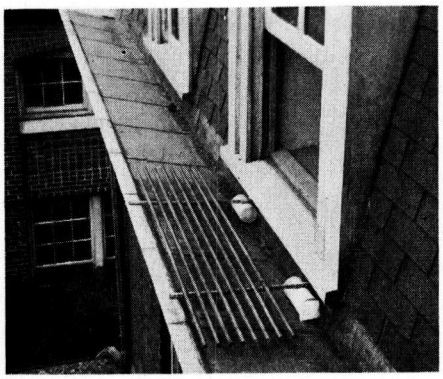


Figure 2. Exposure of Bars on West Ledge of Laboratory of Mechanics at third floor level.

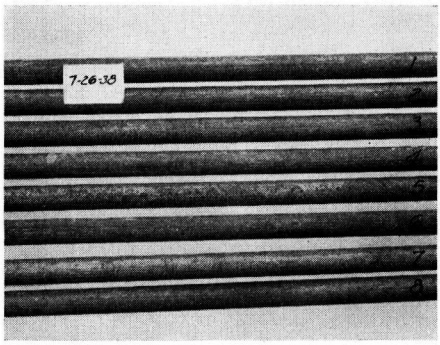


Figure 3. Bars prior to starting Exposure on July 26, 1938

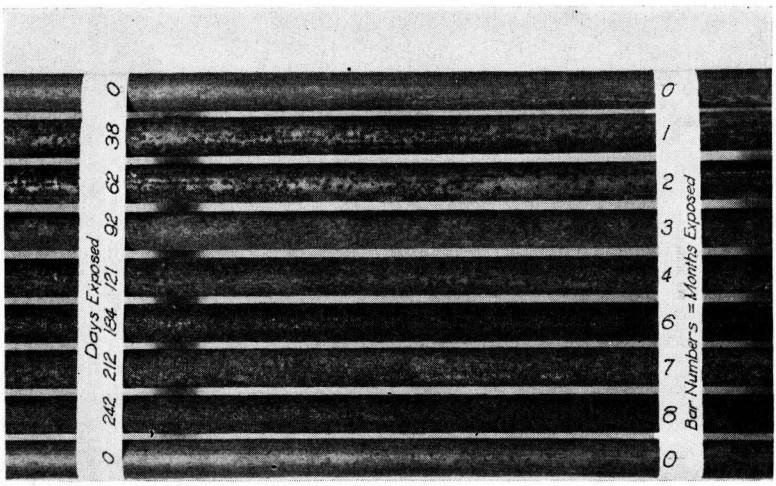


Figure 4. Bars after Exposure. Outside Bars are Unexposed Controls. Bar No. 5 Missing

No. 8, had had, at the end of the period, an outdoor storage of eight months which included the months of August 1938 to March 1939.

The rusted surface of Bar No. 5 (5 months exposure) was inadvertently all bars (except No. 5) after exposure, the two outside bars of the photograph being the two unexposed control portions of the DB length of No. 1 which are numbered 0. The reserve supply, exclusive of the 12-in. tensile specimens,

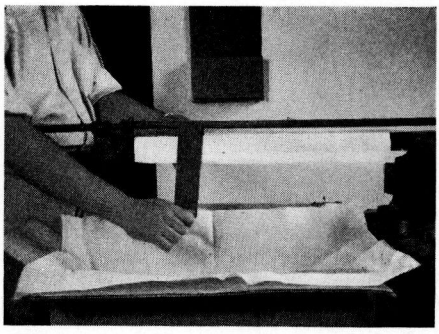


Figure 5. Emery Cloth removal of Rust for weighing. From two 12-inch lengths of each Bar.

was repeated. A new strip of emery cloth was used for each 12-in. length. The details of the process had been evolved by trial to secure rubbed surfaces virtually free of rust and scale. The amount of rust and/or scale secured from the two 12-in. lengths of bar was determined by weighing both the receiving paper and the emery cloth on a pair of analytical balances, before and after the treatment. As shown by Figure 6, a measurable amount of scale was secured from the unrusted bar. The rust secured from the other bars increased in more or less direct proportion to the period of outdoor ex-

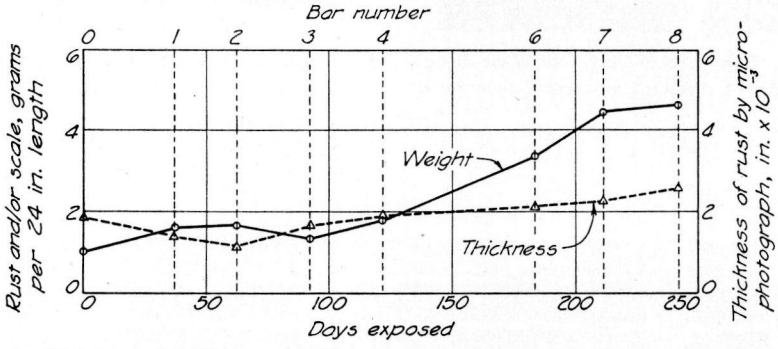


Figure 6. Weight and Thickness of Rust vs. time exposed. Note: Rust on bars 6, 7 and 8 was loose and flaky and some rust is known to have been lost from the photomicrographic specimens in transit and handling prior to securing the photomicrographs.

comprises enough bars to duplicate the series if desired.

MEASUREMENT OF AMOUNTS OF RUST

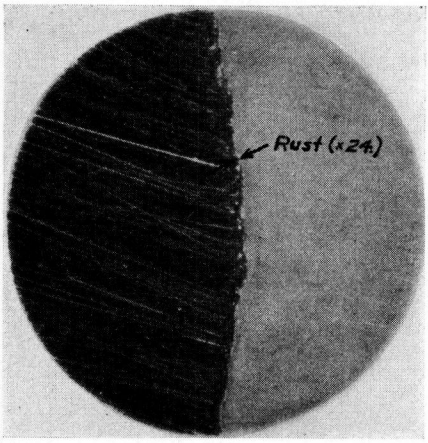
To answer Questions 2 and 3 means had to be devised for measuring or estimating amounts of rust. Two techniques were used: (a) Each 54-in. length of bar was mounted as shown in Figure 5 and two 12-in. lengths near the ends were given 100 passes with a 2 by 10 in. strip of No. 100 emery cloth, all debris (rust and/or scale) being carefully collected on a paper as indicated. The bar was then rotated $\frac{1}{3}$ of a turn and the process

posure after about three months. During the first three months the combined weight of rust and scale remained nearly constant. The authors had not expected to find such an excellent progression and easily definable rate of rust formation as these tests disclosed. (b) The 2-in. museum specimen lengths (GH of Fig. 1) were sent to Ivan Racheff, Testing and Metallurgical Engineer of Chicago, for added study and analysis. Cross-sections were photographed at 24 diameters (Figs. 7 and 8). From these were measured the approximate depth of the rust layer then present. These are plotted on

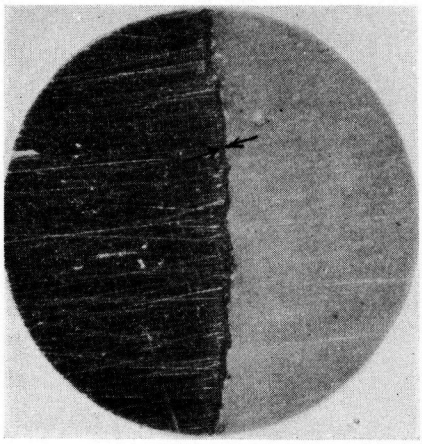
Figure 6 and show the same general trend of consistent increase with added exposure as was shown by the rust-removal tests. Had there not inevitably have been some loss of rust from the flaky surfaces of samples 6, 7 and 8, the trends of the curves would be in still better agreement.

Obviously a third method would have been to weigh equal lengths of bars before exposure and after rust removal but for the conditions of these tests it was more satisfactory to weigh the rust removed as per method (a).

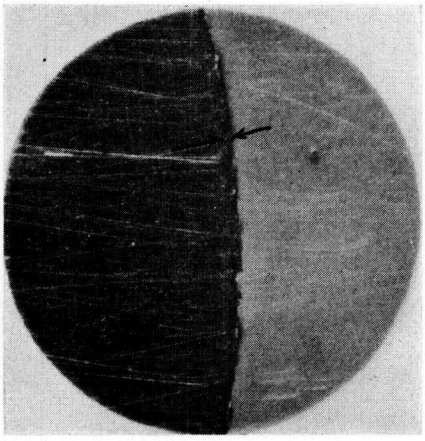
The process of rust formation was noted rather carefully. Very soon after exposure the bars developed a film of loose red rust which came off readily on the hands or any other contacting surface. This rust seems to exert little or no significant influence on the bond. Evidently it is dispersed as the concrete is



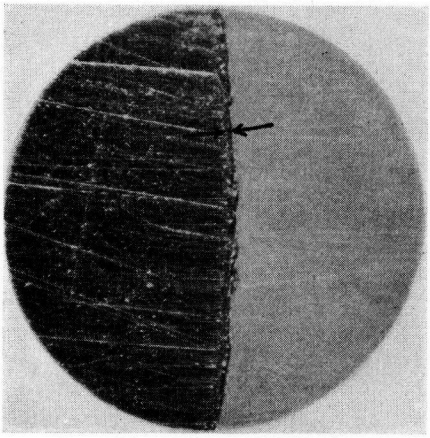
Specimen No. 0



Specimen No. 1



Specimen No. 2

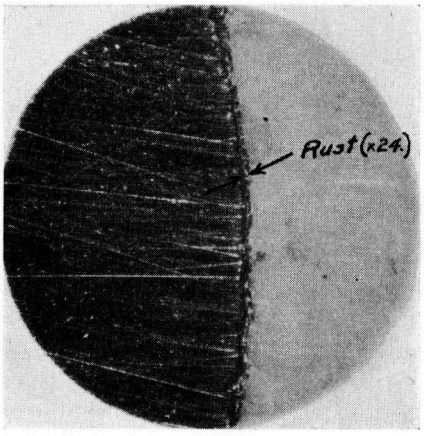


Specimen No. 3

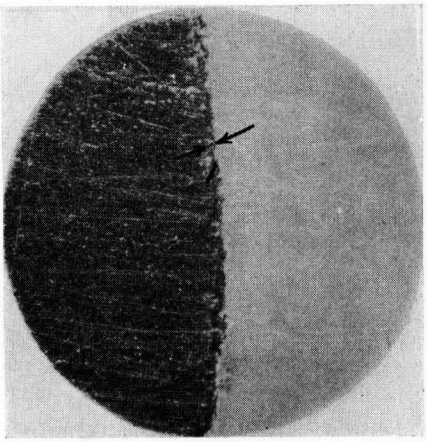
Figure 7. Depths of Rust. Photomicrographs (× 24). Specimens No. 0, 1, 2, 3

placed, and the thin film of first-stage rust is not germane to the problem. At one month the rust was mainly of this type.

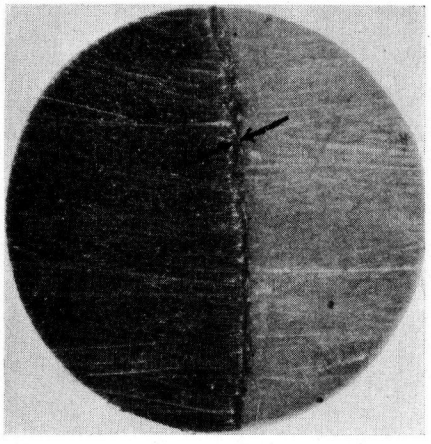
While the rust increased visibly from



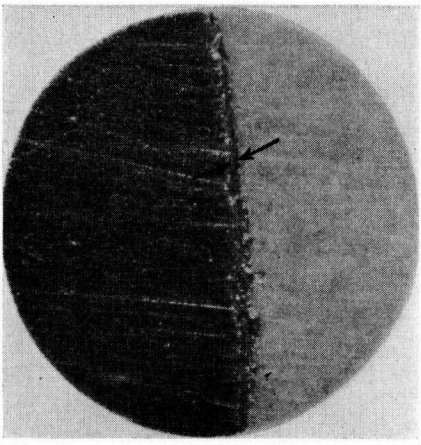
Specimen No. 4



Specimen No. 6



Specimen No. 7



Specimen No. 8

Figure 8. Depths of Rust. Photomicrographs (× 24). Specimens No. 4, 6, 7, 8

the beginning of the exposure as is apparent in Figure 4, the weight of combined mill scale and rust secured was about constant for all bars up to three-months exposure, as shown by the plotted weights on Figure 6. Weather may also have influenced the rate of rust formation and be responsible in part for the faster rates at the greater ages.

The layer of rust increased in thickness but remained quite firm for exposures up to 6 or 7 months, after which the outside portions began to get flaky and loose, being pushed off as the newer layers formed below.

The eight-months overall exposure period proved to be excellent for securing

a good range of rust formation. The type and thickness of the rust layer at eight months had passed anything that would or should be tolerated on a job without some treatment for getting rid of the scaly loose outer layer.

REDUCTION IN EFFECTIVE AREA OF BAR

The amount of rust formed is not a usable indication of the amount of metal destroyed because of differences between the compositions and densities of rust and steel. A thick layer of rust is formed from a very thin layer of steel. The most direct methods for determining reductions in cross-sectional areas would have been:

(a) Careful micrometer measurements of diameters at identical sections before exposure started and again after rust had been removed.

(b) Highly refined weighings of representative lengths of bars before exposure and again after rust removal.

Unfortunately the authors were thinking initially in terms of surface textures and the tests were not so planned as to include either of the techniques indicated above. The oversight was not serious since it was still easily possible to secure good evidence on the relative importance of area reduction in these tests.

All of the bars used in pullout specimens (none of which had been overstrained or damaged as previously pointed out) were recovered from the encasing concrete after test. After the rust had been removed from 10-in. lengths of bar careful micrometer measurements were taken at one-inch intervals along the bars. Table 1 summarizes the data from these measurements. The apparent reductions in diameters and area due to rust are of the same order as the normal deviations in cross-sections along any one bar and the differences due to rust are not, therefore, significant, even for the longest exposures which had heavy layers of loose flaky rust which were objectionable beyond all question.

As an added and very conclusive check on the reduction in area or in load-carrying capacity all bars were tested in tension. The results from these tests (Table 1 Col. (h), (i), (j), (k)) lead to the same conclusion, as above, that within the scope of these tests there was no significant inroad upon the effective area of any bar. The differences in strength between different bars exceed by many times the maximum difference that could be attributed to area reduction by rust. No significant reduction because of rust is apparent even between different specimens from along the same bar.

SURFACE WIPING OF RUSTED BARS (BURLAPPING)

Because of the uncertainties of rust effects, there has been in the absence of evidence, much difference of opinion regarding possible safe minimum requirements relative to the use of badly rusted bars in concrete. Sometimes contractors have been required to wire-brush bars at a considerable cost and in other cases a firm wiping off with burlap has been considered sufficient.

For these limited tests it seemed best to investigate the simplest device that appeared to have a reasonable chance of adequacy.

On the "a" portions of the AG lengths (Fig. 1) the rust was left undisturbed as the bar was cast with reasonable care into a pullout specimen. The "b" portions of the HC length were firmly wiped with a piece of burlap before being cast into pullout specimens. Figure 9 shows the relative appearance of the "a" and "b" bar surfaces.

FABRICATION AND TESTING OF SPECIMENS

With the elimination of the 5-month exposure bars there remained to be fabricated 14 pullout specimens from rusted

bars (wiped and unwiped) and two controls No. 0 (unexposed and unwiped) making sixteen 4 by 4 by 10-in. pullout specimens in all as shown along with the compressive controls in Figure 10.

The specimens were cast vertically from one machine-mixed batch of concrete of proportions 1:2.7:1.6 by weight with a water-cement ratio of 0.55 by

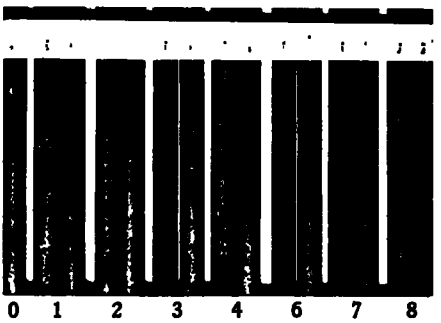


Figure 9. Burlapped Bars (b) vs. those on which Rust was undisturbed (a). Burlapped Bars at right in each case.

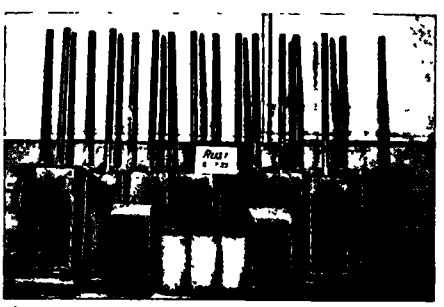


Figure 10. Specimens: Pullouts and Compressive Controls

weight and a slump of 6 in. Hawkeye cement and Des Moines River sand and gravel of $\frac{3}{4}$ in. maximum size were used. The mean 28-day standard-cured compressive strength of the concrete was 4430 lb. per sq. in., the prisms and cylinders giving exactly the same mean strength.

Specimens were standard-cured and tested moist at 28 days, the pullouts in a 20,000-lb. hand operated Olsen testing machine. Five Federal dials (1/10,000 in.) were arranged as shown in Figure 11 to secure data on the first slips, both at the loaded and the unloaded ends of the bar. The ultimate bond resistance and the drag resistance were also observed. First slip was taken as the load at a movement of two dial divisions (0.0002 in.) at the unloaded end and drag resistance was the load after a slippage of 0.08 in. At the loaded end slippage

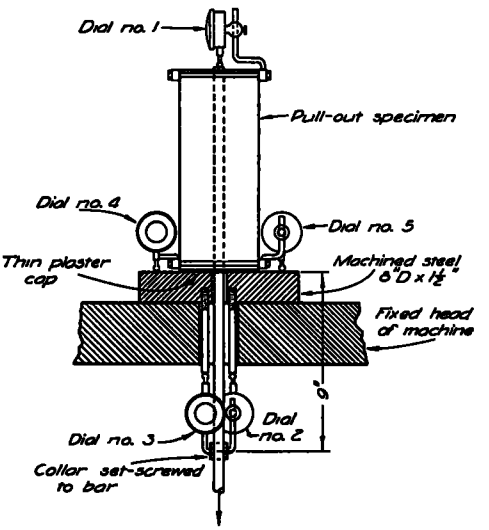


Figure 11. Arrangement of Federal dials for measuring slippages at loaded and unloaded ends of Bar.

started at the first increment of load which agrees with Menzel's observations (6).

RESULTS OF PULLOUT BOND TESTS

Figures 12 and 13 show the load-slip data secured from the tests. The two specimens No. 0 were from the same bar and received identical treatments. The 10 per cent difference between the bond strengths developed is to be explained only on the basis of incidental or experimental variations.

The differences between the curves for the undisturbed and those for the burlapped bars of all the other specimens are in a graduated sequence and are, therefore, seemingly significant even though in amount they scarcely fall without the range of difference between the two identical specimens of No. 0.

Figures 12 and 13 show that the loaded end of the bar slipped from 0.005 to 0.008 in. before "first slip" occurred at the unloaded end. Obviously the relative movement of the two ends of a bar represents the elongation of the steel plus the shortening of the concrete within the embedded length. After slippage became

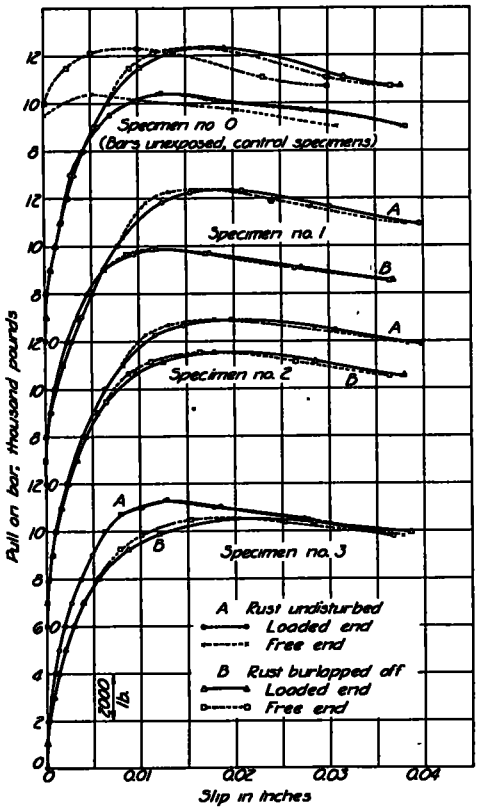


Figure 12. Load-slip curves, specimens No. 0, 1, 2, 3

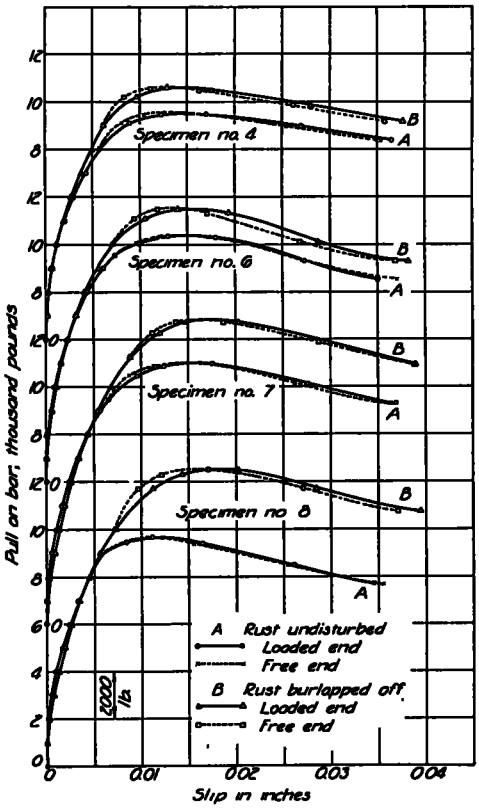


Figure 13. Load-slip curves, specimens No. 4, 6, 7, 8

For exposures up to three months the bars with rust undisturbed exceed in bond developed those which were burlapped but the excess decreases with period of exposure. At about three months exposure the curves cross and the burlapped bars give the higher bond strengths. At eight months the burlapped bars are nearly 20 per cent stronger in bond than are the bars with rust undisturbed.

general (after "first slip") the curves show that the subsequent movements at the two ends of the bar were identical, within the accuracy of measurement. This evidence indicates that there was no apparent change in the nature of the gripping after "first slip" occurred. Had there been an appreciable release or letting go, within the embedded length, the slippage differential would have de-

creased and, had there been a building up of resistance, the slippage at the loaded end would have gained upon that at the unloaded end.

supply a more complete picture than has been generally available heretofore. For the top curve of Figure 12 (No. 0) the curves for slippage at the unloaded end

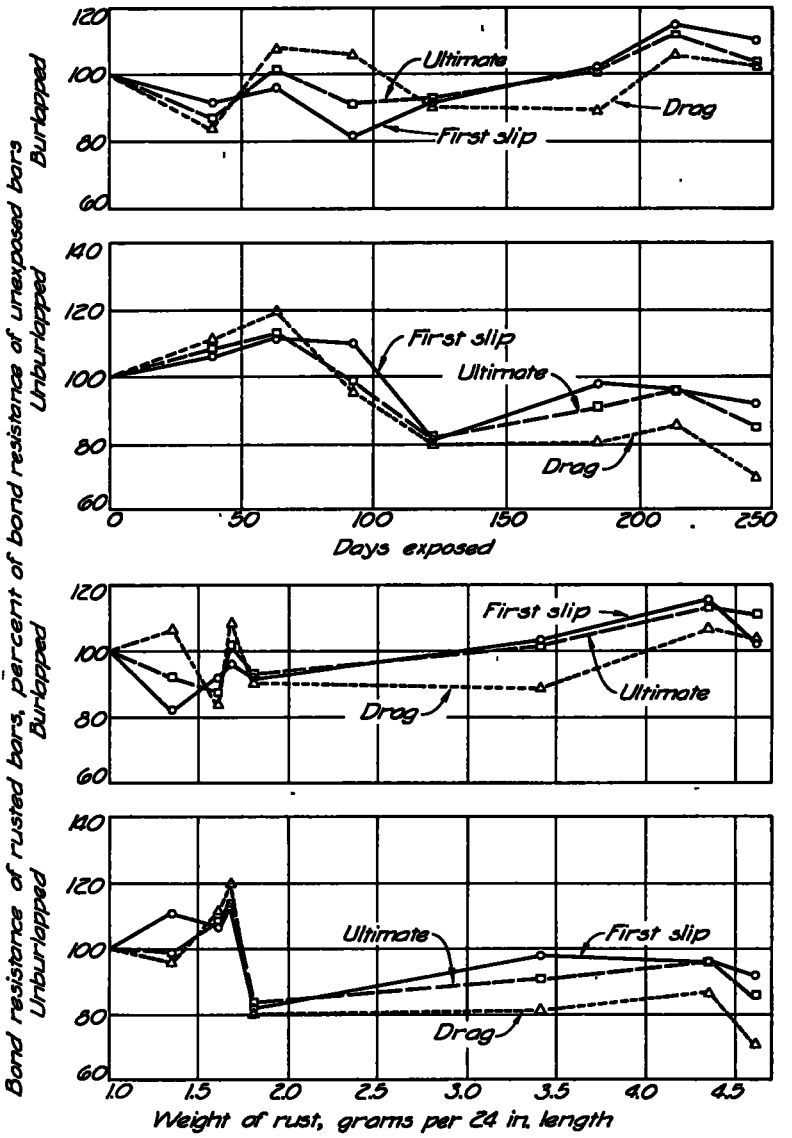


Figure 14. Bond resistance vs. days exposed and weight of rust for both undisturbed and burlapped bars

Previous pullout bond investigators have, with the exception of Menzel (6) secured slip data only on the bar at its unloaded end, and Figures 12 and 13 are shown twice; plotted from the axis of zero slippage as well as alongside the curves for the slippage at the loaded end.

Figure 14 shows the variation of bond

at first slip of unloaded end, at ultimate and shows drag resistance all expressed as percentages of the mean bond resistance developed by the unexposed bars. These are plotted against the number of grams of rust that were secured and also against the number of days of exposure

two specimens for no exposure) the individual deviations tend to obscure possible or probable trends, but the plotted data of Figures 14 and 15 do seem to indicate rather definitely that for exposures beyond three months (90 days):

1. The bond resistance developed by

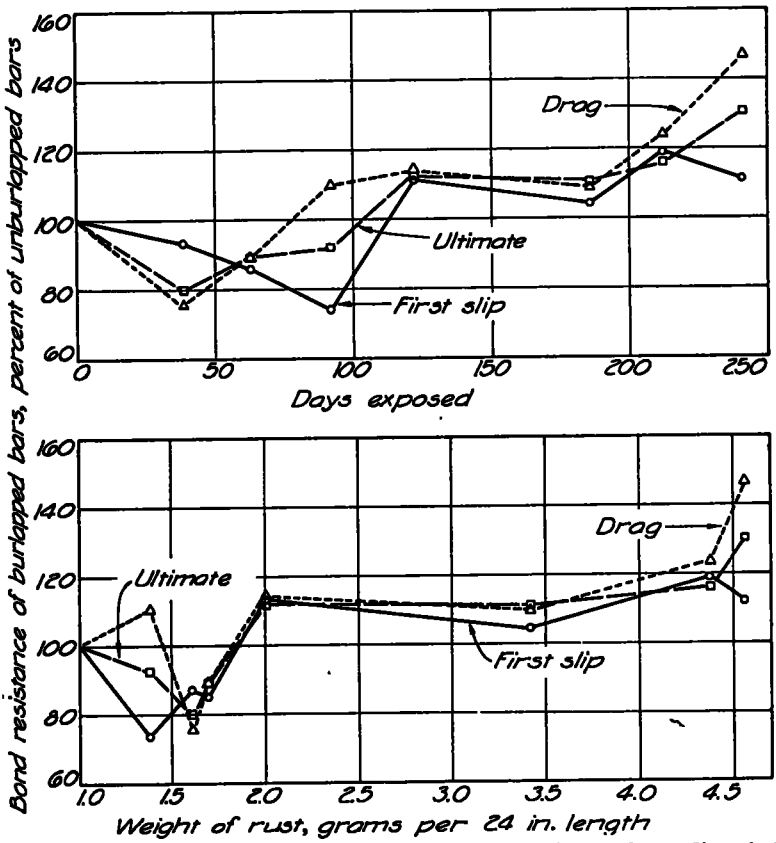


Figure 15. Bond resistance of burlapped bars as percentage of that for undisturbed bars vs. days exposed and weight of rust

for both the undisturbed and the burlapped bars.

Figure 15 is similar to Figure 14 except that the bond resistance developed by the burlapped bars is shown as a percentage of that developed by the bars on which the rust was undisturbed.

Because of the reconnaissance nature of the tests (there being but one pullout specimen of a kind per exposure period; the unburlapped bars (Fig. 14 second curves from top) is somewhat below that developed by the unexposed bars. There is no well defined trend, the first slip and ultimate resistance offered by the six, seven and eight-month exposure specimens all being above that of the four-month specimen, which appears to be out of line. Disregarding the four-month indication, there is a gradual and fairly

well defined downward trend from about a 10 per cent increase at two months to about a 10 per cent decrease at eight months. Relative values for first slip and ultimate agree closely and drag resistance displays a similar trend.

2. For the burlapped bars (top curves of Fig. 14) the trends seem to be submerged within a plus or minus 10 per cent zone of fluctuation for exposures up to three months. From three to eight months there appears to be a rather definite upward trend seeming to indicate that simply wiping off firmly with burlap becomes increasingly effective as the bars become most deeply corroded. For even the worst corrosion (six, seven and eight months exposure) the burlapping gave surfaces which developed resistances equal to or in excess of the resistances of the unrusted bars.

3. It seems, therefore, that in spite of the undulations of the curves, the trends indicate that as the rust becomes deep and flaky, there will be some lowering of the bond resistance (restricted to about 10 per cent within the range of these tests) but that the simple inexpensive expedient of wiping the loosest of the rust off firmly with a piece of burlap can be relied upon to restore the surface to a condition where it will develop equal or greater bond resistance than it would have developed in the unrusted condition.

4. Referring now to the upper curves of Figure 15 and disregarding just now the fluctuations for exposures of less than three months, the beneficial effect of wiping off the loose layers of heavy rust is shown to be consistent and pronounced.

5. Comparing the two upper sets of curves of Figure 14 and the upper set of Figure 15 *for exposures under three months,* it seems likely that the short-time exposure effects are submerged within the range of the experimental and incidental fluctuations to be expected between individual specimens and that neither the exposures nor the burlapping exercised any significant influence upon the results secured. From Figure 14 the mean bond resistances for the burlapped and unburlapped bars slightly exceed the 100 per cent datum of the unexposed bars. By referring to Table 2 one sees that the two rust controls, the unexposed bars, differed from one another by percentages approaching the total spread or range of variation within the three-month exposure period.

From the preceding there is nothing to indicate that firm rust is detrimental to bond and in so far as the results prove anything beyond this, they tend to support the previous reconnaissances of Withey (1), Abrams (2) and the Lehigh University tests (5) that firm rust is beneficial to bond. These tests do demonstrate, moreover, that mere wiping off with burlap will more than restore the bond of the unrusted bar where the rust is deep, loose and flaky.

CONCLUSIONS

1. For securing varying degrees of rust the outdoor exposure was satisfactory and the rust increased rather uniformly with the period of exposure in spite of wide variations in weather during the 8-month exposure period.

2. Removing the rust with the emery cloth and weighing it carefully was satisfactory as a method of rust measurement. Estimated relative areas of rust from photomicrographs of bar cross-sections gave parallel indications.

3. The differences in degrees of rust were readily visible and show up quite well in photographs.

4. The light powdery coating of red rust that appeared during the first month seems to be insignificant in its effect on bond and can safely be disregarded.

5. Within the range of experimental variation the effect of rust on bond resistance was unimportant for exposures up to three or four months. At that age the

bars had a heavy coat of firm rust and appeared much rustier than have the bars on any job within the authors' range of observation and experience.

6. For thick coatings of rust, such as those which characterized the exposures of 7 or 8 months, the bond resistance shows some reduction for the undisturbed bars, but rubbing off the loose rust with burlap produced resistances somewhat above those of the unrusted bars.

7. At all ages beyond 3 or 4 months for these tests burlapping improved the bond and it appears to be a proper and ample treatment to apply when there is question of whether or not the coating of rust is objectionably heavy or loose.

8. The indications from these tests are reassuring in that any reasonable amount of fairly firm rust on reinforcing bars is not cause for alarm. Even heavy coatings can easily be rubbed off to the point where they too need give no concern.

9. Rust is much lighter and bulkier than steel and in spite of loose thick flaky rust after seven and eight months exposure the actual reduction of cross-section was inconsequential and could scarcely be detected either by micrometer measurements or by tensile strength tests.

10. The conclusions from these tests are not out of line with those secured from a contemporary series conducted on deformed reinforcing bars at Lehigh University (5).

11. After the slippage of the bar of a pullout specimen becomes general (after "first slip," so-called) the amounts of additional slippage at the loaded and unloaded ends are equal.

ACKNOWLEDGMENTS

The authors are indebted to the Laclede Steel Company of St. Louis for the rail steel that was used in the tests. Mr. W. A. Jennings of the Economy Form Corporation of Des Moines, supplied the steel forms which were used for casting all of the specimens except the cylinders. The authors are also indebted to Professor W. M. Dunagan and Dr. Glenn Murphy, co-workers in the Department of Theoretical and Applied Mechanics, for advice and assistance of various kinds.

The work reported herein has been conducted as one phase of a joint cooperative program between the Highway Research Board, R. W. Crum, Director, and the Iowa Engineering Experiment Station, Dean T. R. Agg, Director.

REFERENCES

Note: Rather complete lists of references on bond and related researches are given in connection with chapters previously published in the Proceedings of the Highway Research Board as follows:

Vol. 14 (1934) p. 271-283, references and digests (Chapter II).
Vol. 16 (1936) p. 92-95, references and digests (Chapter V).
Vol. 17 (1937) p. 186, references, (Chapter VIII).
Vol. 18 (1938) p. 129, references, (Chapter IX).

References No. 4, 5, and 6, below, have not appeared in any of the lists of references given above.

1. M. O. Withey. "Tests on Bond between Concrete and Steel in Reinforced Concrete Beams," Bulletin 321 (1909) University of Wisconsin, (p. 46-47).
2. D. A. Abrams. "Tests of Bond between Concrete and Steel," Bulletin 71 (1913) University of Illinois Engineering Experiment Station, (p. 47-48).
3. Gilkey, Chamberlin and Beal, "The Bond between Concrete and Steel," *Proceedings* American Concrete Institute, Vol. 35 (1938-39) (Journal Sept. 1938) p. 1-20. (Rust on p. 10-11; added references on p. 19-20).
4. J. R. Shank. "Effect of Bar Surface Conditions in Reinforced Concrete," *Engineering Experiment Station News,* Ohio State University, Vol. VI, No. 3, June 1934 (p. 9-12).
5. A. E. Lindau. "Report on Rusty Bar Bond Tests and High Yield Point Steel Research," at 1939 Annual Meeting of Concrete Reinforcing Steel Institute. Preliminary report on bond tests conducted by Lyse, Johnston and Cox at Lehigh University on rusty deformed bars 1937-1939.
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STUDIES OF SUBGRADE PRESSURES UNDER FLEXIBLE ROAD SURFACES

By A. T. Goldbeck

Engineering Director, National Crushed Stone Association

SYNOPSIS

This paper shows the results of pressure measurements taken under 4 in., 6 in. and 8 in. of stone screenings placed on a subgrade and subjected to 4,000, 8,000 and 12,000-lb. wheel loads, in the laboratory of the National Crushed Stone Association.

Load-indentation tests were also made on the subgrade in place as well as on the surfacing material, in an effort to tie in the present tests with subsequent investigations. This report is merely one of progress, but it is felt that the results may be useful in the later formulation of a theory for flexible road design.

It is the purpose of this paper to present the results of some preliminary tests made to determine the subgrade pressures under flexible road surfaces of different thicknesses. The results have not reached the stage which will permit of generalizing them into a theory for flexible road design, but, none the less, light is thrown on distribution of pressure to the subgrade and on the effect of differences in thickness and rigidity of the flexible surface on subgrade pressure distribution.

METHOD FOR MEASURING PRESSURES

The tests were conducted in the laboratory making use of a bin constructed of heavy timber backed with I-beam supports. The base of the bin was designed for rigidity. The soil pressure measurements were made by the use of a soil pressure cell such as described by the author before the last annual meeting of the Highway Research Board.[1]

In the bin was first placed a sand-clay mixture which had previously been mixed with the desired amount of water to give it the necessary degree of stability. If a soft subgrade was desired, additional water was used in the mix. The subgrade was rolled in 3-in. layers by the use of a sheep's foot roller made with concrete disks alternating with disks having a toothed circumference. This roller was followed by a heavy hand concrete roller of cylindrical form. The characteristics of the subgrade materials used in these tests are shown in Table 1 by a series of laboratory tests made through the courtesy of the American Bitumuls Company.

It was decided that the most convenient surfacing material to use in these preliminary tests would be stone screenings, for they could be very readily mixed with water and laid in a homogeneous layer and could be well compacted in the laboratory bin with the method just described. The present tests were all made with stone screenings as the surfacing layer, laid to different thicknesses and allowed to dry to different amounts depending upon the rigidity desired. When first laid and rolled, the wet screenings did not have a great deal of stability, but, as they dried they attained high stability and apparently even had bending resistance until cracks were formed under the test load.

The method of installing the soil pressure cell and its location were subject to change as experience was obtained during the progress of the tests. It is recognized that the soil pressure cell, having a thickness of about 1½ in. and having rigidity, theoretically is not an ideal device for this particular kind of pressure measure-

[1] *Proceedings*, Highway Research Board, Vol. 18, Part II, p. 66 (1938).

ment, because it introduces a rigid element into the subgrade structure which has possibilities of producing error. These possibilities were fully recognized before the tests were begun but it is believed that with the methods used, an accuracy has been attained which is in keeping with the kind of measurements taken and the use for which they are intended.

The soil pressure cell was installed in the center of the bin, in all cases with the weighing face down in contact with the subgrade. In some cases the cell was first laid on the subgrade before applying the screenings and rolling them. In other cases, especially where unstable screenings were used in the test, the cell was placed with its weighing face $1\frac{1}{2}$ in. below the surface of the subgrade. In this type of installation, however, the cell was surrounded with a brass collar loosely filled with a graphited gland packing so as to eliminate friction between the sides of the cell and the surrounding collar. See Figure 2. In theory the unit pressure on the weighing face of the cell would be the unit pressure at the base of the cell which was set level with the surface of the subgrade.

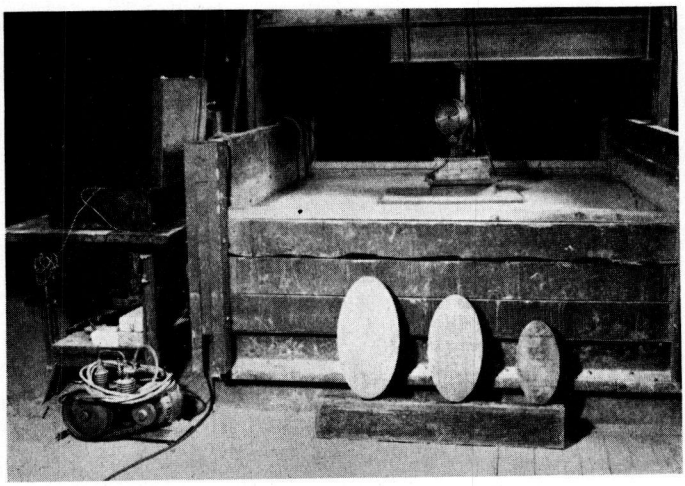


Figure 1. Apparatus for Measuring Subgrade Pressure

Here is shown the 6 ft. square by 3 ft. high bin with the hydraulic jack mounted for applying load. The different size elliptical bearing blocks correspond with the contact areas produced by 4,000, 8,000 and 12,000 lb. wheel loads. On the left is the pressure gauge box to which is connected a soil pressure cell mounted on the subgrade under the stable surfacing material.

METHOD OF LOAD-APPLICATION AND LOAD MEASUREMENT

Loads were applied through elliptical bearing blocks designed to have an area equal to the area of contact of tires suitable for 4,000, 8,000 and 12,000 lb. loads respectively. The dimensions of these blocks and of the unit pressures equal to the appropriate tire pressures and the areas of contact are given in Table 2.

In all cases the bearing block was first set on a $\frac{1}{2}$-in. rubber pad placed on the wearing surface. A hydraulic jack was used for applying the load. This jack was calibrated frequently to insure continued accuracy.

The procedure in taking measurements

TABLE 1

CHARACTERISTICS OF SCREENINGS SURFACE

MATERIAL

Limestone screenings

GRADATION

Sieve	Percentage passing
¼-in.	100
No. 10	82.4
No. 200	23.9

STABILITY (PUNCHING SHEAR TESTS)

Moisture, %	1.9	3.4	6.5	9.68
Punching shear	5300	4500	2700	3400

Screenings were compressed at moisture content of 6.5 per cent, then air dried to respective moisture contents.

CHARACTERISTICS OF SUBGRADE

MATERIAL

Sand-clay mixture

GRADATION

Sieve	Percentage passing
¼-in.	100
No. 10	99
No. 200	36.2

STABILITY TEST

Moisture, %	3	5	6	7	9	10	12
Stability (½-in. bottom layer)	40,000+	40,000+	29,000−	15,500	7500	2900	0

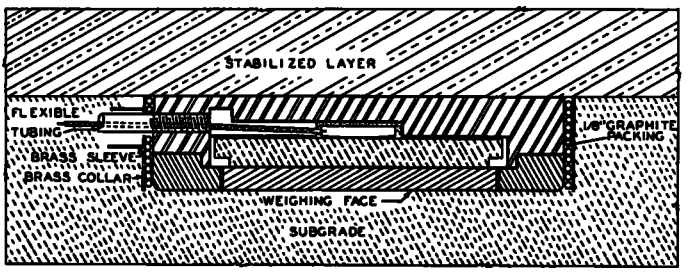


Figure 2. Soil Pressure Cell

was to place the bearing block at the side of the bin and pointing toward the cell; load was applied and pressure readings taken on the cell. Zero load subgrade pressure readings were made before and after each load application. The block was then advanced a short distance and was again loaded and the pressure readings taken. This procedure of loading and advancing the block was repeated until the block was centered over the cell. In this manner an influence line for subgrade pressure was obtained and the pressures would thus represent the increase in pressure at any given spot on the subgrade as the load was advancing toward that spot. This is the condition of loading which actually exists as a truck wheel operates on a road surface. It would seem reasonable to believe that this influence line for pressure likewise very closely represents the actual distribution of longitudinal pressure. It is believed that substantially the same results are obtained by this method of loading by the use of a single cell as would be obtained were a number of cells installed to determine the pressure distribution.

The method used has the advantage of subjecting the subgrade to advancing pressures such as would obtain under actual loading conditions. From our tests we know that the mere fact of having loaded the surface in a given zone, results in the compaction of the subgrade in that zone. This compaction increases the subgrade resistance in the zone which has been loaded as compared with the unloaded zone.

In a manner similar to that just described, lateral distribution of pressure was obtained, the bearing block in this case being advanced laterally, rather than longitudinally.

LOAD INDENTATION CHARACTERISTICS OF SUBGRADE AND OF SURFACING MATERIAL

A series of subgrade pressure-distribution tests would not be complete unless they were supplemented by tests to determine the respective bearing values of the subgrade and of the surfacing material. It seems reasonable to assume that if the maximum pressure on the subgrade exceeds the load-supporting value of the subgrade when the load is applied, then that particular combination of surfacing material and subgrade is in danger of failure. It becomes highly important, therefore, to determine the load-supporting value of the subgrade in order to determine if the maximum subgrade pressure is approaching or even exceeding the subgrade supporting value. Undoubtedly, the surfacing material gives some degree of confinement to the subgrade and, furthermore, it would seem desirable to make a load-supporting value test on the subgrade in its undisturbed condition. Having these two points in mind it was decided to make load-indentation tests on the subgrade in the following manner.

The design of the apparatus and its load-carrying capacity as a whole was such that a bearing block of 100 sq. in.

TABLE 2

DATA ON BEARING BLOCKS

Long diameter, 2 A (in.)	Short diameter, 2 B (in.)	Area, sq. in.	Total load, lb.	Unit load, lb. per sq. in.
12-1/8	6-1/16	58	4,000	69.0
15-3/4	7-7/8	97	8,000	82.5
18-1/8	9-1/16	130	12,000	92.3

area seemed appropriate. A hole approximately $\frac{1}{2}$ in. greater in diameter than the diameter of the block was very carefully formed in the surfacing material and after very carefully scraping the subgrade material to a level, flat surface, the steel bearing block was mounted on it and load was applied very slowly by the hydraulic jack. Simultaneously, indentation readings of the bearing block into the subgrade were taken. The first tests were performed by the continuous application of load and by the measurement of indentation through the use of a steel scale. But as refinements were developed in the method, load was applied in increments and after each increment of application, a period of one minute was allowed to elapse and indentation readings were taken by the use of two Ames dials reading to 0.001 in. In this way a load-indentation curve was obtained and since the time intervals are alike the respective curves on different subgrades should be comparable.

In a similar manner load-indentation tests were made on the surfacing material and these tests may be taken as a measure of the physical properties of this material. In the case of the surfacing materials, however, a bearing block having only 10 sq. in. was employed, because were a large block to be used, the subgrade would contribute to the indentation. In this test it was desired that the indentation be confined entirely to the surfacing material if possible. By the use of this 10-in. bearing block, this objective was in large measure accomplished.

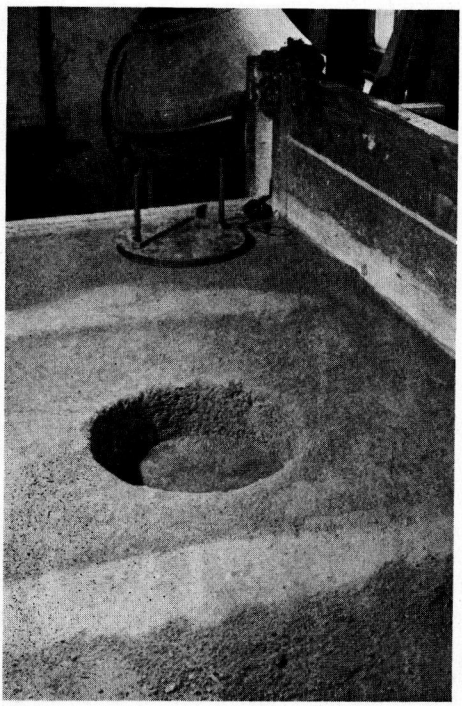


Figure 3. Hole dug through screenings layer preparatory to making load-indentation or bearing value test on subgrade.

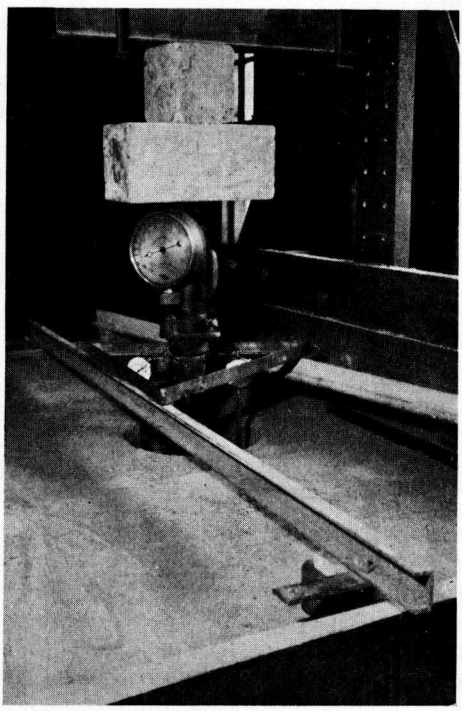


Figure 4. Making load-indentation (bearing value) test on subgrade. Bearing block, circular, with 100 sq. in. of area. Indentation is determined with two dials reading to 0.001 in.

It no doubt would be desirable if additional tests could be made in the nature of an extrusion or punching shear test, especially if the samples could be taken from the subgrade and surfacing materials respectively in their undisturbed condition. In the absence of these, however, it is felt that the present load-indentation tests will serve a very useful purpose in identifying and relating sub-

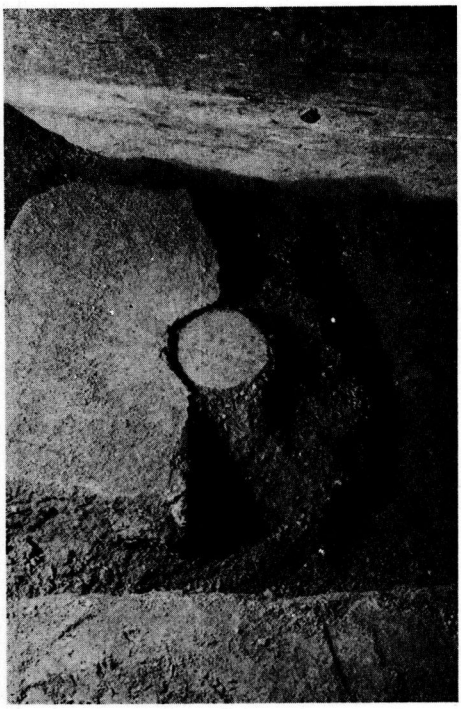


Figure 5. Appearance of screenings layer after making load-indentation test; circular bearing block, 10 sq. in. in area. Note the surface along which shearing action occurred during the test. Through the underlying cone the load is carried to the subgrade.

sequent tests to those at present described. All of the pressure readings will not be given at the present time, but merely typical curves to show some of the extremes of pressure obtained.

CURVES SHOWING TEST DATA

The following curves of test data have been selected from the tests thus far made in order to bring out the influence on maximum pressure and pressure distribution of variables in the thickness and rigidity of subgrade and wearing surface. Also, they show the influence of the magnitude of load applied.

In this test shown in Figure 6 the screenings were 4 in. thick, had only 3.4 per cent moisture and were very stable. The subgrade had 9.4 per cent moisture and it likewise was quite stable. Four thousand and 8,000 lb. wheel loads were applied on their respective bearing blocks

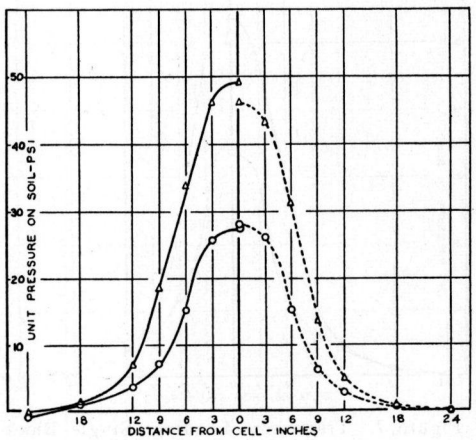


Figure 6. Longitudinal Loading, Single Block

Total load	Unit load p.s.i.
○—4,000	69
Δ—8,000	82.5

Test No. 2:
4-in. Screenings on 18-in. Subgrade
Moisture: Screenings....3.4
Subgrade.....9.4

with the resulting pressure distribution shown. Roughly, 48 lb. per sq. in. was the maximum pressure on the subgrade under the 8,000 lb. wheel load and 28 lb. per sq. in. under the 4,000 wheel load.

The lateral pressure distribution for the above described conditions is shown in Figure 7. It will be recalled that the bearing block is advanced transversely in making these tests for transverse distribution and in view of the fact that a center zone of pressure is created by

virtue of the prior application of the bearing block, in all probability there is a higher concentration of pressure on this center zone than in those portions of the subgrade to which direct application of pressure has not been applied. It has been noted time and again that in those spots in the subgrade which have received prior compression in excess of that received in other spots, there is a tendency for higher pressure concentration in the compressed portion. This fact

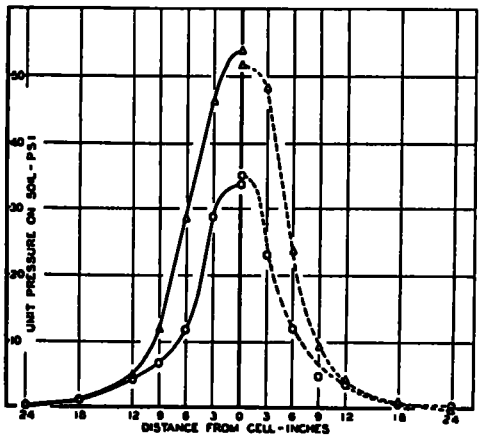


Figure 7. Transverse Loading, Single Block

Total load	Unit load p.s.i.
○—4,000	69
△—8,000	82.5

Test No. 2:
4-in. Screenings on 18-in. Subgrade
Moisture: Screenings....3.4
Subgrade.....9.4

makes it extremely difficult to obtain closely concordant check readings, but at the same time it demonstrates the lack of necessity for extreme accuracy in making tests of this character. We are dealing with a plastic material in both the subgrade and the wearing surface, or at least an inelastic material, and pressure conditions are changing rapidly during the making of the test. In Figure 7 it will be noted that the highest lateral pressures are spread out less than the high longitudinal pressures in the vicinity of the bearing block where most of the pressures are concentrated.

In Figure 8 are shown typical load-indentation tests on the subgrade. The stepped shape of the curve is due to the interval of one minute after applying the load, during which indentation of the bearing block is taking place. The slant portions of the curve represent the inter-

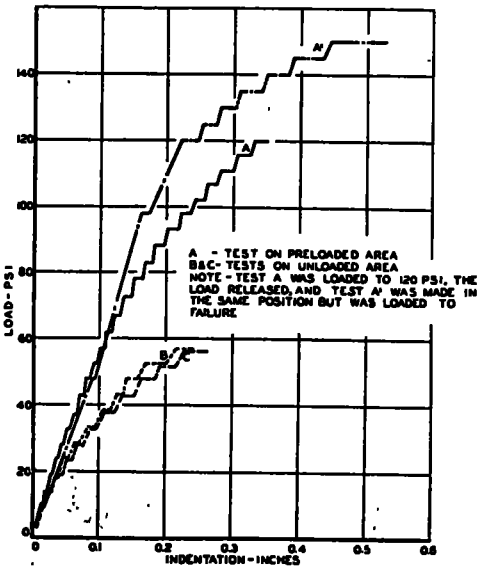


Figure 8. Load Indentation Test on Subgrade

Test No. 2:
4-in. Screenings on 18-in. Subgrade
Moisture: Screenings....3.4
Subgrade.....9.4

vals during the application of the load increments. The load increments are applied in a fraction of a minute, but not in a definite time interval. Curves B and C were obtained on those portions of the subgrade which had not been loaded by the bearing block while tests A and A' were made on the central portion of the bin where the bearing block had been previously applied in making the load-distribution tests. Thus, the mere fact of loading the surface has increased the

resistance of the subgrade to a very great extent. This test indicates that the subgrade is improved by the action of traffic, due no doubt to its greater compaction. This compacting action of traffic has of course been observed also in practice.

Note that there is a more or less definite yield point beyond which the subgrade yields at a higher rate. It would seem reasonable to use curve A as the curve which represents the significant pressure-indentation characteristics of the subgrade, for it probably would more nearly approach the condition of a subgrade in service after having been subjected to traffic. If this assumption is valid, then it would seem that the bearing value of this particular subgrade might be taken at around 65 lb. per sq. in., this point being that beyond which the curve deviates rather rapidly from the straight line.

In Figure 9 are shown typical load-indentation tests on the screenings surface. Since this particular surface contained only 3.4 per cent of water, the screenings were very stable as shown by the fact that rapid yielding did not take place until after a load of approximately 350 lb. per sq. in. was applied. It is of course somewhat indefinite as to what point on the load-indentation curve may be considered as the bearing value of the material. The selection of this point needs discussion and clarification. Perhaps it should be selected at a definite indentation, such as 0.1 in.

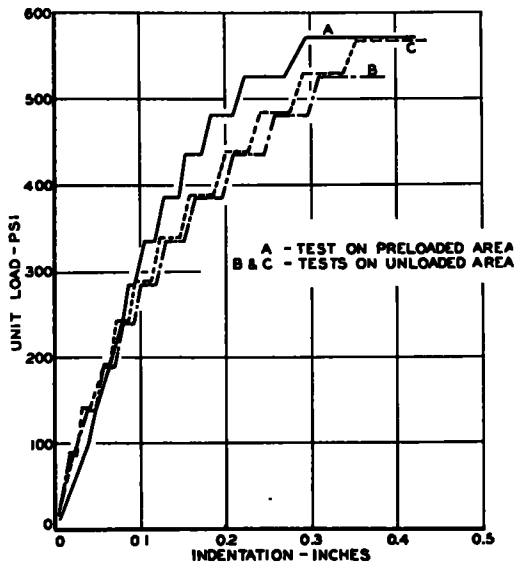


Figure 9. Load-Indentation Test on Screenings

Test No 2:
4-in. Screenings on 18-in. Subgrade
Moisture: Screenings....3.4
Subgrade.....9.4

Figure 10 is to be compared with Figure 6. The moisture conditions in the screenings and subgrade are not exactly alike, but they are closely comparable. Here, the surfacing material was 6 in. thick instead of 4 in. as in Figure 6. It will be noted that the maximum subgrade pressures are reduced. It was found possible to apply a 12,000 lb. wheel load on

the 6-in. material without failure, whereas only 8,000 lb. could be applied in the case of the 4-in. layer.

In Figure 11, 8 in. of stabilized screenings were used instead of 6 in. as in Figure 10 and 4 in. as in Figure 6. Here, again, a very great reduction in the subgrade pressures is noted. The pressure under the 12,000 lb. wheel load in Figure 11 is no greater than that due to the 8,000 lb. wheel load in Figure 2.

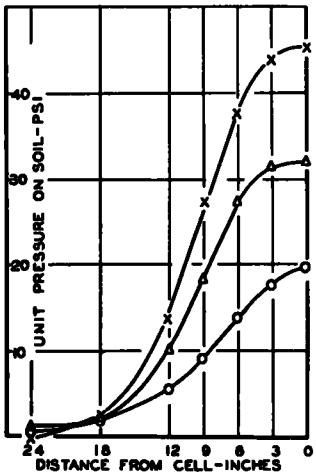


Figure 10. Longitudinal Loading, Single Block

Total load	Unit load p.s.i.
○—4,000	69
Δ—8,000	82.5
×—12,000	92.5

Test No. 1:
6-in. Screenings on 18-in. Subgrade
Moisture: Screenings....3.8
Subgrade.....9.2

In contrast with the prior results, the tests shown in Figure 12 indicate the high pressures which are possible when the surfacing layer is unstable. Here, the screenings layer has 6 per cent of moisture and it will be noted that under the 8,000 lb. wheel load, something over 70 lb. per sq. in. maximum pressure was attained.

The influence of the high per cent of moisture on the stability of the screenings is well shown in Figure 13. Note that the bearing value of these screenings is only about 100 lb. per sq. in. as con-

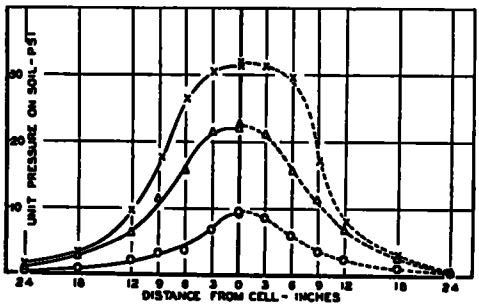


Figure 11. Longitudinal Loading, Single Block

Total load	Unit load p.s.i.
○—4,000	69
Δ—8,000	82.5
×—12,000	92.5

Test No. 6:
8-in. Screenings on 18-in. Subgrade
Moisture: Screenings....3.5
Subgrade.....8.3

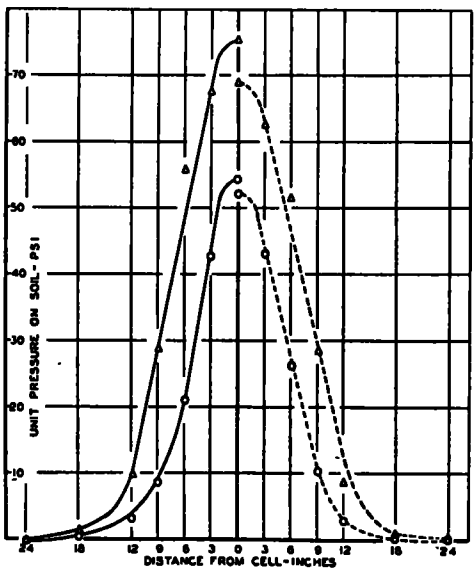


Figure 12. Longitudinal Loading, Single Block

Total load	Unit load p.s.i.
○—4,000	69
Δ—8,000	82.5

Test No. 9:
6-in. Screenings on 18-in. Subgrade
Moisture: Screenings....6.0
Subgrade....10.0

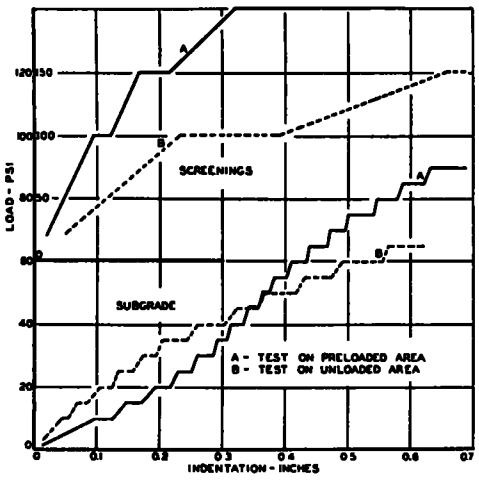


Figure 13. Load-Indentation Tests

Test No. 9:
6-in. Screenings on 18-in. Subgrade
Moisture: Screenings....6.0
Subgrade....10.0

trasted with 350 lb. per sq. in. with a low percentage of moisture. It is also seen that the subgrade in this particular case was none too stable. At least it showed a high indentation under the application of load.

The results, as far as maximum pressure intensities under different wheel loads are concerned, are summed up in Figure 14. The ordinates are the maximum pressures obtained on the subgrade under the center of the wheel load. A very great contrast in these pressure intensities is evident. For illustration, in test No. 3, made with 4 in. of screenings having a high percentage of moisture, a 4,000 lb. wheel load shows a maximum pressure intensity of well over 70 lb. per sq. in. This probably is in excess of the bearing capacity of the subgrade. When

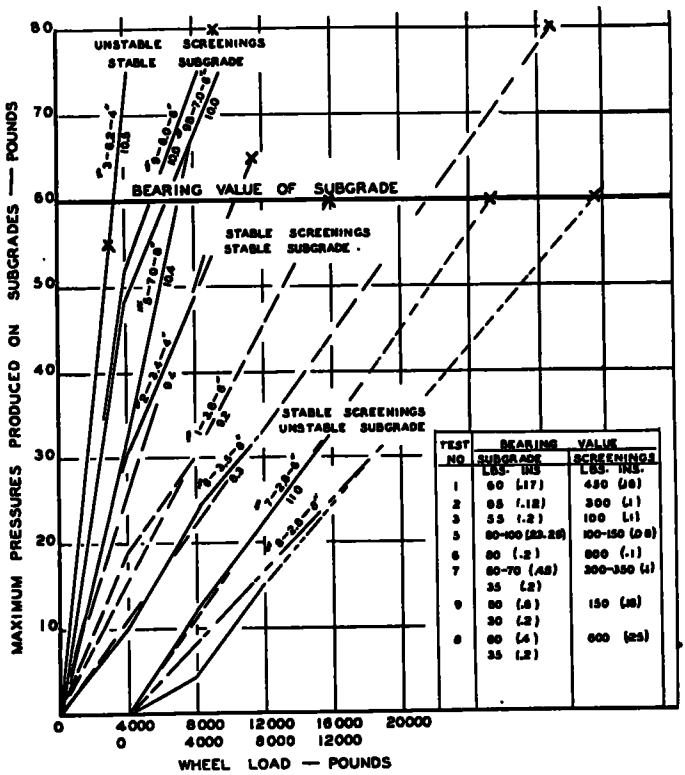


Figure 14. Maximum Subgrade Pressure Obtained Under Various Conditions

the screenings layer was stabilized by drying it out to only 3.4 per cent moisture, the pressure intensity on the subgrade due to a 4,000 lb. load was reduced to only 30 lb. per sq. in., well within the bearing capacity of the subgrade. It is indicated that this layer, with an 8,000 lb. wheel load had a maximum subgrade pressure of only 49 lb. per sq. in., apparently also well within the bearing capacity of the subgrade. The effects of other combinations of wheel load, surfacing, thickness and condition of the subgrade are shown in Figure 14.

CONCLUSIONS

Most of the theories for the design of the flexible road have assumed that the subgrade pressure is uniform in intensity and that it occupies an area which is confined within lines sloping outward from the area of surface contact, generally at 45°. Some of the older theories even ignore the fact that there is an area of contact between the tire and the road. It is quite evident that these theories are fallacious for the pressure intensity is not uniform.

No attempt is made at the present time to formulate a theory of design. It is too early for that, but rather are the present tests aimed at obtaining results which may later be of value in the formulation of a theory. It is hoped that ultimately the tests will be made to include different thicknesses of typical flexible surfaces and typical subgrade materials. It is also proposed to pursue them in the field if possible.

The present measurements take no account of impact and field tests should be made to determine to what extent, if any, impact is an important factor. For this purpose different apparatus or a modification of the present apparatus will be necessary. The whole project of flexible road design is important enough to warrant cooperative tests to the end that more rapid progress might be made.

PRESENT DESIGN PRACTICE AND CONSTRUCTION DEVELOPMENTS IN FLEXIBLE PAVEMENTS

By B. E. Gray

Chief Engineer, The Asphalt Institute

SYNOPSIS

The term "flexible pavements" might properly include quite a variety of surfacing materials, but for purposes of this paper they are considered as those bituminous types of surfaces which are mudless and dustless at all seasons of of the year.

As a result of questionnaire study to the various State highway departments, some interesting conclusions have been reached.

The importance of the design of subgrade is becoming generally recognized and that the correct method of determination of thickness of pavement should be predicated upon bringing subgrade values up rather than attempting to build massive bases down.

The development of types of surfaces has gone through a certain cycle and is rather intimately related to the development of equipment.

The influence of traffic under load distribution has greatly modified the design of pavements and has made possible the effective carrying of traffic under volumes which would have been destructive if the balloon tire had not been invented and become universally available on all motor driven equipment.

The selection of type of surface also should take into account all aggregate available locally. Cognizance should be taken of the fact that stone and gravel vary in size according to geological formations, and surfaces should be designed so as to use most economically the complete plant output. An illustration of this better use of plant output, and at the same time vastly improving the road structure, is the growing use of stone screenings of approximately 1½ inches thickness as an insulation layer between the natural subgrade and the pavement proper.

Methods for accurately evaluating subgrade values are still needed. Several States have methods of making such tests which apparently give good results. One interesting development is the attempt of one State to design the thickness of base courses in accordance with the soil characteristics, such as plastic limit, liquid limit, etc.

An encouraging note from this review of practice is the intensive attention which is being given to the design of the subgrade, which after all must carry the load, and which until recent years has been almost entirely overlooked as an element in pavement design.

There have been several papers presented in recent years on the design of flexible pavements and reference is made to some of them in the references appended to the end of this discussion. These studies have taken up various aspects of the problem ranging from the design of paving mixtures themselves to formulae for stress distribution. While untreated gravel and stone layers are often thought of as flexible type pavements, the modern conception includes only types which are mudless and dustless at all seasons of the year. Hence, this discussion will apply solely to the bituminous types of surfaces, with untreated gravel and stone considered only as foundation courses.

The matter of stress distribution will not be touched upon in this paper; (1) because a great deal of what might be said has already been covered in previous papers, notably the one by Mr. Benkelman before the 17th Annual Meeting of the Highway Research Board in 1937, and (2) because there are other studies currently being carried on, one by Mr. Goldbeck and another by Prof. Spangler. The Asphalt Institute contemplates a research beginning this coming year on still another phase of this study.

The design of flexible pavements, how-

ever, has been much influenced by a number of collateral factors which but recently have been taken into account, and a brief discussion of some of them may be of interest. The so-called flexible types of pavement form today the larger portion of the highway system of the country. According to April, 1939 issue of *American Highways,* such surfaces number 176,818 miles out of a total of 274,830 miles of all weather surfaces on the state systems, or 64 percent.

In order to ascertain the trends of present construction practice, a questionnaire was addressed to the state highway departments. The replies are shown in Tables 1 to 6 in such detail as space will permit. The comments accompanying the replies indicate that a very definite technique of foundation design is developing. The older practice of making heavier pavements to care for bad soil conditions is giving way to the cheaper and better practice of building subgrades having high bearing power. This is the outstanding development of recent years and while it is required for all types of pavements, the benefits show up markedly in the flexible type because thickness may be reduced in almost direct proportion to subgrade improvement.

SUBGRADE DESIGN

Of the several methods, the first is densification of the natural soils themselves. The effect of such procedure is often the equivalent of several inches of pavement. Many states now call for subgrades to be consolidated to 90 percent or 95 percent density as measured by the Proctor test. The paper by Ira B. Mullis, entitled "Principles Applying to Highway Road Beds," appearing in the September, 1938 *Proceedings,* American Society of Civil Engineers, is worthy of careful study. As a result of wide study of surface failures, Mr. Mullis states that "no case (failure) is recalled in which the bulk density of the *roadbed* was in excess of 1.50."

$$\text{Bulk density} = \frac{\text{dry weight in grams}}{\text{bulk volume in cu. cms.}}$$

In other words by building subgrades to densities maintainable the year around above this figure no pavement troubles occur.

One state, Florida, definitely calls for a subgrade of 30 lb. minimum bearing power. Several states require that soils of the A-4 to A-7 class be covered with some pervious material or be in some way modified. Arizona suggests a new method of determining base thickness according to soil characteristics. This is shown in Table 7.

The state authorities frankly say that Table 7 is to be taken as a guide only, awaiting longer experience.

Minnesota has a real frost problem, and in Table 8 is shown their design procedure.

For many years the writer has called attention to the wasteful practice of unclassified excavation, whereby the weathered top soils are so frequently thrown away and the unstable subsoils are left as the subgrade on which a pavement is to be placed. Most of the ills which have surrounded pavement construction of all types have been due to subsequent subgrade movement which could be largely prevented by selecting material from weathered soils. Inasmuch as these soils are relatively cheap, a layer of 12 in. or more in depth can be placed for a fraction of the cost of a pavement and yet the effect is equivalent to a substantial thickness in pavement. Selected subgrades having high stability, of course, lead to thinner surface sections and resultant economy and freedom from maintenance.

An interesting report is noted from Texas where tests are made to detect suitable deposits of materials for subgrade construction. This describes "the

THICKNESS OF SURFACE AND BASE FOR ROAD MIX (REMAINDER REQUIRED [illegible])

State	Thickness of surface	Type of base	Thickness of base	Subgrade treatment	Remarks
	In.		*In.*		
Alabama	2	Clay gravel, top soil, sand clay—chert.	6	None—which was a mistake	Not used at present—but a good type.
Arizona	2 to 3	Rock—gravel	0 to 15	According to soil properties	See Table 6.
Arkansas	1 (average)	Sand clay gravel, crushed stone, W. B. macadam.	6 to 10	Satisfactory soil layer	Additional base when needed.
California					
Colorado					
Connecticut					
Delaware	2	Traffic bound slag, gravel or stabilized soil.	8		
District of Columbia					
Florida	1½	Sand clay or Florida limerock	6 to 8	30 lb. bearing value soil	Used as retread.
Georgia	2	Top soil, sand clay pebble soil	8 in. stabilized	Where needed	Not much used.
Idaho					Not used.
Illinois	2½	Gravel or crushed stone	6		
Indiana	2½ to 3½	W. B. macadam stabilized base or old pavements.	6 to 8	Drainage with 6 in. open joint tile.	Base receives bit. prime coat, and aggregate back fill where needed.
Iowa					None constructed recently.
Kansas					
Kentucky	2	Stone base or stabilized base	4 to 8	Type base varies with subgrade	Base course 2 ft. wider than surface.
Louisiana					
Maine					
Maryland	2	Macadam or old pavements	8 for new macadam	2 screenings stabilized	Base sealed with bit prime coat.
Massachusetts	2	Gravel	6 to 12		
Michigan	2	Macadam or gravel	6 to 8	Bad soil removed—sand sub-base.	Mostly county projects.
Minnesota	2				Only one job laid this type.
Mississippi					None.
Missouri					
Montana					
Nebraska					
Nevada					
New Hampshire	3	Selected gravel	6 to 9	Stabilized with gravel.	
New Jersey	1½ to 2	Macadam or gravel	4 to 6	None.	
New Mexico	1½ to 2	Crushed or pit run rock, gravel, caliche	6 to 8	Material manipulated, watered, rolled to secure full compaction.	
New York	2½	Bituminous treated stone	Usually old macadam widened		Do not use.
North Carolina					
North Dakota					
Ohio	2½	Old macadam	6 to 9		Sometimes 3 to 5 traffic bound base.
Oklahoma					
Oregon					
Pennsylvania	2	Napped stone or commercial stone	6 to 8	Unsuitable subgrade material removed—base thickness increased.	
Rhode Island	2½	Water bound macadam	5½	Careful drainage, extra foundation where needed.	
South Carolina					
South Dakota	2	Crushed gravel or rock, (stabilized)	4 or more	Rolling	Base sealed with road oil.
Tennessee					
Texas					Not used.
Utah				Bituminous prime coat.	
Vermont	2½	Gravel	12 to 24	As needed	Mixed surface treatment.
Virginia	1¼	Traffic bound macadam or crushed or screened gravel.	6-8 or 10-9-10		
Washington					Do not construct.
West Virginia	1 to 1½	Stone or slag	8 in two courses	Extra base and stone drains; sometimes creek gravel for base.	
Wisconsin					
Wyoming	1½ to 2	Crushed rock or gravel	Varies with soil condition	95 percent maximum density compaction.	

TABLE 2

Thickness of Base and Surface for Road-Mix (Dense Graded Type)

State	Thickness of surface	Type of base	Thickness of base	Subgrade treatment	Remarks
	In.		*In.*		
Alabama	2	Clay gravel, top soil, sand clay, chert	6	None, which was mistake.	Base too thin—type not used today.
Arizona	2 to 3	Rock—gravel	0 to 15	According to soil properties	See Table 7.
Arkansas	2¼ (average)	Sand, clay, gravel—crushed stone—W. B. macadam.	6 to 10	Satisfactory soil layer—or additional base.	
California	3	Crusher run stone or gravel or imported material.	6 (average)		
Colorado	1½	Gravel or crushed rock	6 or more	If necessary.	
Connecticut					
Delaware					
District of Columbia					
Florida	6	Natural soil	None	Subgrade of 30 lb. value or more.	Sand mixtures.
Georgia	2	Top soil, sand clay, pebble soil	8 (stabilized)	Where needed	Not much used.
Idaho	2½	Crushed gravel or rock	6 to 12		
Illinois	2½	Gravel or crushed stone	6		Base receives bituminous prime coat.
Indiana	3	Crushed stone or gravel	3	Careful drainage, use of 6 in. open tile drains aggregate back fill.	
Iowa					
Kansas	2¾ (average)	Stabilized soil, soil aggregate mix, pit run aggregate.	4 to 12	Fills carefully placed to 90 percent Proctor.	Selected borrow 12 in.-18 in. depth.
Kentucky					
Louisiana					
Maine	1½	Gravel	12 to 30		
Maryland	4 to 6	Natural soil	None		Sand mixtures.
Massachusetts					
Michigan	2½	Gravel	6 to 8	Bad soils removed, sandy soils replaced, mostly county projects.	
Minnesota					Not laid—see Table 6.
Mississippi	6	Selected sand clay	12 to 15	None	Carefully controlled gradation.
Missouri	1½	Clay bound aggregates	4	4 in. depth bituminous stabilization.	
Montana	1½ to 2½	Crushed aggregates, 100 percent pass 2½, 25-50 percent pass ¼ in.	8 to 24 loose	Not over 12 percent 200 mesh in base course.	
Nebraska	1¼ to 3 (1⅞ average)	Natural subgrade		Over A-2 to A-5 soils only	Also a 5 in. sand mix in sandy soils.
Nevada	2½	Graded gravel, crushed or uncrushed	4 to 8	Base increased on poor soils	Complete soil analysis made.
New Hampshire		No surfaces originally built this now 2 to 3 in. thick.	type, but many miles surface	treatments with sand cover are	
New Jersey					None.
New Mexico	2 to 2½	¾ by 2 in. aggregate	8 (two courses)	Material manipulated, watered, rolled to secure compaction.	
New York					
North Carolina	¾ to 2				Used to resurface old roads only.
North Dakota	1½	Stabilized gravel	4 to 6	Compaction with sheepsfoot roller.	
Ohio	2	Old macadam	6 to 9		Sometimes 3 to 5 in. traffic bound macadam.
Oklahoma	1½	Gravel or crushed stone	6 or more		
Oregon	2½	Pit run or screened gravel	6 to 12	None	Base thickness determined by conditions.

TABLE 2—Continued

State	Thickness of surface	Type of base	Thickness of base	Subgrade treatment	Remarks
	In.		*In.*		
Pennsylvania	2	Napped stone or commercial stone	6 to 8	Unstable soils removed, or stabilized—increased base thickness.	
Rhode Island	2	Gravel	6	Drainage and extra foundation where necessary.	
South Carolina	2	Crushed rock or gravel (stabilized)	4 or more	Base courses receive bituminous prime and used a year before surfacing.	
South Dakota					
Tennessee					
Texas	1½ or more	Gravel, caliche, stabilized soil	6 or more	Stabilized where needed with selected soils or oil.	
Utah	1½ to 2½	Gravel	12 min.	Materials consolidated as per AASHO. 1½ in. surface on secondary roads.	
Vermont	2½	Gravel	12 to 24	Bituminous prime coat.	
Virginia	1¼	Water bound macadam	10-9-10	Where needed, 6 to 10 in. selected material.	
Washington	2	Crushed rock or selected sand gravel with rock top.	12 to 20	Place layer fine granular material.	Base thickness according to subgrade.
West Virginia	1½	Stone or slag	8	Extra base, stone drains where needed; sometimes creek gravel bases.	
Wisconsin	2	Gravel	4 to 6	Where required 12 to 24 in. selected material.	
Wyoming	1½ to 2	Gravel or crushed rock	According to soil and moisture	95 percent maximum density compaction.	

TABLE 3

Thickness of Base and Surface for Penetration Macadam

State	Thickness of surface	Type of base	Thickness of base	Subgrade treatment	Remarks
	In.		*In.*		
Alabama					Do not use.
Arizona					
Arkansas					
California	1½ and 2	Crusher run stone, gravel or imported material.	Average 6		
Colorado					
Connecticut	2½	Crushed stone	7	Gravel sub-base where needed.	
Delaware					
District of Columbia					
Florida					None constructed recent years.
Georgia	3	Top soil sand clay, pebble soil	8 (stabilized)		Little used today.
Idaho					Not used.
Illinois	6	Earth subgrade	None	Receives bituminous prime coat.	
Indiana					See Table 5.
Iowa					
Kansas					None constructed recently.
Kentucky					
Louisiana					
Maine	3	Crushed stone	7, 5, 7	12 to 24 in. gravelly sub-base	
Maryland	3	Crushed stone, filled with clean screenings	5	2 in. screenings stabilized	Special care in filling base voids.
Massachusetts	2½	Crushed stone, sand filled	3½ to 4½	6 to 12 in. gravel.	
Michigan					None built.
Minnesota					Not laid.
Mississippi					None.
Missouri	1 and 2½	Water bound macadam	7½ and 8	1½ in. limestone screenings	Shoulders are oiled.
Montana	1 to 1½	Crushed aggregates	8-24 loose	Selected soils	2 lift heavy surface treat.
Nebraska					
Nevada					
New Hampshire	3	Crushed stone	4	Stabilized with gravel.	
New Jersey	3	Macadam	4	None.	
New Mexico					See Table 6.
New York	3	A pen. macadam base over sand filled macadam.	3 to 4 and 3 to 4 respectively	6 to 12 in. gravelly sub-base.	
North Carolina					Do not use.
North Dakota					
Ohio	3	Water bound macadam or old pavement	7	1½ in. granular insulation course.	
Oklahoma					
Oregon	¾-1¼-3	Crushed rock or gravel	6 to 18	None.	
Pennsylvania	3	Napped or commercial stone	6 or 8 or 10	Bad subgrade removed or stabilized—base thickness increased.	
Rhode Island	2½	Water bound macadam	5½	Drainage	Extra foundation where needed.
South Carolina					
South Dakota					
Tennessee					
Texas					
Utah					
Vermont	3	Gravel or stone	12 to 24	Broken stone base course on sub-base.	
Virginia					
Washington	2½ to 3	Crushed rock or selected sand gravel with rock top.	12 to 20	Sealing with layer fine granular material.	
West Virginia	3	Stone or slag	8 (2 courses)	Extra base course where needed—also stone under drains.	
Wisconsin					None built.
Wyoming					Not used.

TABLE 4

THICKNESS OF BASE AND SURFACE FOR COLD-LAID BITUMINOUS CONCRETE

State	Thickness of surface	Type of base	Thickness of base	Subgrade treatment	Remarks
	In.		*In*		
Alabama	1½ to 2	Clay gravel, top soil, sand clay, chert.	8	As much as 10 in. topping over A-7 soils.	This is coming resurfacing type.
Arizona					
Arkansas					
California	See Table 6				See Table 6.
Colorado					
Connecticut	2	Stone	variable	Usually employed resurfacing over old pavements.	
Delaware	2	Macadam or portland cement concrete.	8		Surface placed in two courses.
District of Columbia					
Florida	1½ or 3	Lime rock or portland cement concrete.	6 or 8	30 lb. bearing value, 3 in. depth laid in two courses.	
Georgia	2 binder, 1½ top	Pebble soil, gravel, stone or p. c. concrete.	6 to 8	Where needed—also 5 and 6 in. sand bituminous mix.	
Idaho					See Table 6.
Illinois	2	On old or new portland cement concrete.	6		Prime coat usually omitted.
Indiana	1½ to 2	Water bound macadam or stabilized aggregate.	6		Plant mix open graded aggregate type.
Iowa	2	Soil-aggregate or rolled stone	5	All fills placed 6 in. layers top 6 in. sheepsfooted to 95 percent Proctor.	
Kansas					County use only, but state plans use.
Kentucky					
Louisiana					
Maine	2	Bituminous concrete	3	12 to 24 in. gravel—also used over 8-6-8 in. P. C. C.	
Maryland	2 (average)	Old road or p. c. concrete	8	1¼ to 1½ in. binder course, ½ in. top plus dressing.	
Massachusetts	2¼	Hot penetration macadam	4	6 to 12 in. gravel.	
Michigan					None built.
Minnesota	2	Over old portland cement concrete.			Very little used.
Mississippi					
Missouri					
Montana					
Nebraska					
Nevada	2½	Graded, crushed or uncrushed gravel.	4-8 coarser grading in deep section.	Thickness base varied. A plant mix similar to dense road-mix.	
New Hampshire	3	Broken stone or crushed screened gravel.	4	Stabilized with gravel	Liquefier type surface.
New Jersey	Type a—1 over 2 pen. macadam. Type b—½ to 1 top over 1½ to 2 binder.	Macadam or p. c. concrete	4 macadam, 6 p. c. concrete	None—1.65 gal. per yd. used in 2 in. pen. course.	
New Mexico					
New York	2	P. c. concrete, penetration, macadam or old pavements.	8 plain, 7 reinforced p. c. concrete; or 8 in. macadam over 6 to 12 in. bank gravel; macadam 2 courses top course 4 in. penetration macadam.		

TABLE 4—Continued

State	Thickness of surface	Type of base	Thickness of base	Subgrade treatment	Remarks
	In.		*In.*		
North Carolina	¾ to 1½	Over old pavements			Used for resurfacing.
North Dakota					
Ohio	2 to 2½	Old pavements or hot mix bituminous concrete.	7	1¼ in. granular insulation course	Hot mix used in widening work.
Oklahoma	1½	Gravel macadam, p. c. concrete.	6 or more		
Oregon					
Pennsylvania	2 and more	Napped stone or commerical stone.	6 or 8 or 10	Remove or stabilize bad subgrade—increase base.	
Rhode Island	2	Gravel	6	Extra thickness where needed—good drainage.	
South Carolina					
South Dakota					
Tennessee					See Table 6.
Texas	1 or more	Pavements of all types	6 min.	Stabilized where needed with selected soils or oil.	
Utah					
Vermont					
Virginia	1½ binder, ¾ top	Plain p. c. concrete or W. B. macadam.	Concrete 8 W. B. macadam 10-9-10	Where needed using 6 to 10 in. selected material.	None.
Washington	1½ to 2½ min.	Old pavements	Variable	This type used almost entirely over old brick, p. c. c., asphalt.	
West Virginia	1 to 1½	Stone, slag or portland cement concrete.	8	Extra base where needed, stone weeper drains.	
Wisconsin	1	Bituminous base	2 to 4	12 to 24 in. selected material where needed.	
Wyoming					Not used.

TABLE 5

THICKNESS OF BASE AND SURFACE FOR HOT-LAID BITUMINOUS CONCRETE

State	Thickness of surface	Type of base	Thickness of base	Subgrade treatment	Remarks
	In.		*In.*		
Alabama	1 to 2 top 3 to 4 binder	Clay gravel, topsoil, sand clay, chert	8	As much as 10 in. over A-7 soils	Selected soil in subgrade.
Arizona					
Arkansas					
California	2	Bituminous concrete	7-5-7 thickened edge.	Selected borrow material	Also used over old pavements.
Colorado					
Connecticut					
Delaware					
District of Columbia					
Florida	1½ or 3	Limerock or portland cement concrete	6 to 8	30 lb. subgrade material.	
Georgia	2 binder	Pebble soil, gravel, stone or portland cement concrete.	6 to 8	Where needed	Not much used.
Idaho	1½ top 2	Alternate design 4 in. bituminous concrete soils not permitted; Compaction to AASHO Spec. M-57.	on 4 in. rock or gravel or 8 to 12 in. rock or gravel, A-5 to A-7		
Illinois	2 (or 3 sheet)	New or old portland cement concrete	6		Prime coat usually omitted.
Indiana	1½	Combination of pen. macadam, W. B. macadam, & coarse stone, in thicknesses 3½, 4¼, 4; 3, 4, 4; and 3, 3½, 2½ respectively. In competition with 9-7-9, 9-6-9, 8-5-8 portland cement concrete.			
Iowa					
Kansas	1½ to 2	P. c. concrete	7	Shaped and rolled	City work.
Kentucky	New specifications call for all bituminous concrete pavement in competition Two layers coarse mix, top layer dense.			with portland cement concrete inch for inch.	
Louisiana					
Maine	2	Bituminous concrete	3	12 to 24 in. gravel	Also used over 8-6-8 in. portland cement concrete.
Maryland	2 to 3	Primed, macadam, old pavements or p. c. concrete.	6 to 8		Surface laid in 2 courses.
Massachusetts	2½	Hot penetration macadam	4	6 to 12 in. gravel.	
Michigan	3½-2½-3½	Gravel	6 to 8	Sand sub-base over bad soils	Base built year before surfacing.
Minnesota	2 to 2½	Portland cement concrete			City construction only.
Mississippi	1½	Bituminous concrete (black base)	2½	Selected sand clay 6 to 12 in. Uses high pen. AC as stability permits.	
Missouri	1¾ or more	Over old portland cement concrete	6		Used largely for resurfacing.
Montana					
Nebraska					
Nevada					
New Hampshire					No new construction.
New Jersey	2 to 3	Gravel or portland cement concrete	6	3 in. laid in two courses 1½ in. binder, 1½ in. top.	
New Mexico					
New York	2 (topeka)	P. c. concrete, pen. macadam, or old pavements.	8 plain, 7 reinforced portland cement concrete; or 8 in. macadam over 6 to 12 in. bank run gravel, macadam laid 2 courses top being 4 in. pen. macadam.		
North Carolina	¾ to 1½				Used for resurfacing, See Table 6.
North Dakota					
Ohio	2 to 2½	Old pavements or new hot mix bituminous concrete.	7	1¼ in. granular insulation course.	
Oklahoma					
Oregon	5	Crushed gravel or rock (See Table 6)	8 to 12	None—Surface 1 in. coarse texture on 4 in. dense binder.	

TABLE 5—Continued

State	Thickness of surface	Type of base	Thickness of base	Subgrade treatment	Remarks
	In.		*In.*		
Pennsylvania	2 and more	Stone and portland cement concrete	6 or 8 or 10	Remove or stabilize, bad subgrade—increase base thickness.	
Rhode Island	3	Portland cement concrete	7	Extra foundation where needed, good drainage.	
South Carolina	1½	Sand clay, top soil, stabilized soil	5 to 8	Base increased over bad soil—heavy s. t. types and thin road mix.	
South Dakota					
Tennessee					
Texas	2 or more	Both flexible and rigid types	6 or more	Stabilized where needed with selected soils or oil.	
Utah	2	P. c. concrete or asphaltic concrete	6 in. p.c. concrete; or 4 in. black base on 6 in. gravel. Blended and consolidated per AASHO spec. Max. dens.		
Vermont					None.
Virginia	1½ binder 1 top	Plain p. c. concrete	8 or 10	Where needed	Sheet asphalt type.
Washington	1½ to 5	Old pavements, new bases, or selected material.	Variable	If new pavement—sealing course of granular material.	
West Virginia					
Wisconsin	3	Portland cement concrete	6 to 8		
Wyoming					Not used.

TABLE 6

THICKNESS OF BASE AND SURFACE FOR MISCELLANEOUS TYPES (SPECIALLY USED BY STATES)

State	Thickness of surface	Type of base	Thickness of base	Subgrade treatment	Remarks
	In.		*In.*		
Alabama	5/8 to 1/8	Clay gravel, top soil, sand clay, chert	8	As much as 10 in. over A-7 soils—combination s. t. and 50 lb. plant mix top.	
Arizona					
Arkansas	50 lb. to 60 lb. stone	Stabilized soil various types	6		Heavy surface treatment.
California	2½ to 3	Crusher run rock or gravel base	3½ to 6	Where needed. A variety of cold laid plant mix.	
Colorado					
Connecticut					
Delaware	1	Traffic bound slag, gravel or stabilized soil.	8		Heavy surface treatment.
District of Columbia					
Florida	¾	Sand clay, lime rock, soil cement	6 to 8	30 lb. bearing value material. Heavy surface treatment.	
Georgia	1½	Top soil, sand clay, clay gravel, or stone.	6 to 10	Where needed. Combination s. t. and 50 lb. plant mix top.	
Idaho	2½	Crushed gravel or rock			Plant mix dense graded aggregate.
Illinois					
Indiana	¾	Penetration macadam, water bound macadam.	8½ to 11½	Drainage with open joint tile—rock asphalt seal courses.	
Iowa	½ to ¾	Soil aggregate or rolled stone	5	All fills in 6 in. layers, sheepfooted top 6 in. layer, heavy s. t. type.	
Kansas	less than one inch	Stabilized soil	4 to 10	Fills compacted, selected soils where needed —surface treatment type.	
Kentucky					Heavy surface treatment.
Louisiana	1	Gravel	6	None	S. t. and premix gravel types.
Maine	2	Gravel	12 to 30		
Maryland					
Massachusetts					
Michigan					Grading gravel ¾ in.—200 mesh.
Minnesota	¾ to 1¾	Stabilized gravel	See Table 8	See Table 8	
Mississippi	2	Sand asphalt base	3	Selected sand clay soil	This is a sand mixture.
Missouri	Armor coat	Clay bound aggregates, W. B. macadam, bituminous soil Stab.	5 to 6	0 to 4 in. stabilization. Used as base for later surfacing.	
Montana					
Nebraska	2 to 3	Stabilized soil	2 to 5 (3 average)	Also experimenting with 4-5 in.; stabilized soil as a surface.	
Nevada					
New Hampshire					
New Jersey					
New Mexico	¾ to 1	Old portland cement concrete or other pavement.	6 to 10		Rock asphalt type.
New York					
North Carolina	1 to 5	Sand asphalt, both hot and cold laid. Hot one course 3 to 5 in.	laid in 2 courses	3 to 3½-in. binder, 1 to 1½-in. top; cold laid	
North Dakota	½ to ¾	Stabilized gravel	4 to 6	Standard compaction sheepsfoot roller.	
Ohio	2 to 3	Old pavements or pre-mix base course.	6 to 9	Bituminous mixtures of less accurate controlled proportioning.	
Oklahoma	1	Gravel macadam	6 or more		Double surface treatment.
Oregon	2	P. c. concrete	6	4 in. crushed rock sub-base—asph. conc. type.	
Pennsylvania	2½ to 3	Portland cement concrete	8 or 9		Sheet asphalt surface.

TABLE 6—Continued

State	Thickness of surface	Type of base	Thickness of base	Subgrade treatment	Remarks
	In.		*In.*		
Rhode Island	2 to 3	W. B. macadam or p. c. concrete	6 to 7	Extra foundation where needed	Variations of plant mix types.
South Carolina	2	Crushed rock or gravel	4 or more	Rolled to optimum	Cold laid plant mix type.
South Dakota					
Tennessee					
Texas					
Utah	2½	Gravel	12 to 24	Materials blended, consolidated AASHO spec for density & moisture content.	
Vermont					
Virginia	¾ approximate	Soil—gravel—sand clay	6 in. sand clay, 14-9-14 others	As needed	S. t. also see Table 1.
Washington	¾	Crushed rock or selected sand gravel with rock top.	12 to 20	Sealing course with granular material—s. t. used a year prior to s. course.	
West Virginia	¾ and ¾	Stone or slag	8 (2 courses)	Extra base where needed—¾ in. leveling course followed later by top.	
Wisconsin					Light surface treatments.
Wyoming	½				

electrical resistivity instrument," and the procedure is a type of geophysical study. The equipment can be set up and readings taken at one foot intervals down to 10 ft. in about 20 min. According to the report the procedure is very successful and permits discovery of deposits which are not available by surface examination and which are difficult and expensive to find by the usual test pit method.

One state in referring to failures of road-mix surfaces some years ago notes that the combination of 6-in. base and no subgrade treatment led to uncertain results, while by improving subgrades according to soil studies and increasing base thickness to 8 in. the greatly increased traffic of present day is carried without failures. The question of improving subgrades by adding granular material has often deterred engineers because of fancied expense. In most parts of the country such material is on the contrary not expensive, and Table 9 is inserted to show just what the various depths do cost.

Numerous studies have indicated that for 10 or 15 cents per sq. yd., the subgrade resistance often can be brought up from 10 lb. under adverse conditions of moisture, to 50 lb. after treatment.

Several states mention the use of an insulation course over the subgrade prior to surfacing. The writer first noticed this procedure some years ago in Ohio. It involves placing from 1 to 2 in. of screenings over the subgrade prior to placing pavement. The value of this layer is not as widely appreciated as should be the case. In the construction of macadam and gravel bases, it is a well known fact that in certain soils a loss of metal is encountered due to the intrusion of the soil into the aggregate layer. Furthermore, obtaining suitable density in the base course is retarded because there is not sufficient resistance in the subgrade against which to roll. By placing this insulation course, a solid subgrade is

TABLE 7 (ARIZONA)

BASE COURSE THICKNESS REQUIRED FOR VARIOUS SUBGRADES AND PROPERTIES OF BASE COURSE MATERIALS

	−40 mesh lineal shrinkage	Centrifuge moisture equivalent	Plasticity index	Material retained on No. 3 sieve	Material passing No. 200 sieve	Thickness base course required
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*In.*
Subgrade	−3	−18	−6	10+	−20	0 to 6
Subgrade	−5	....	−12	10+	−30	6 to 9
Subgrade	−7	....	−15	....	−40	9 to 12
Subgrade	−9	....	−20	....	−45	12 to 15
Subgrade	9+	....	20+	....	45+	15+[2]
Subgrade						100% Passing
Aggregate Base Course	−4	−18	−6	[1]	−20	1 in. Sieve
Select Material	−4	−18	−6	[1]	−20	.....
Imported Borrow	−4	−18	−10	....	....	.....

[1] Amount of material retained on No. 3 varies according to the amount available. Desirable to hold between 25 percent and 50 percent if economically feasible.

[2] Subgrades of this type should not be used in fills in new construction.

N. B. This table taken from paper by W. G. O'Harra, Chemist, Arizona Highway Department, presented at 1939 Arizona Roads and Streets Conference.

TABLE 8 (MINNESOTA)

MINNESOTA STABILIZED GRAVEL BASE DESIGN PRACTICE TO BE SURFACED WITH THREE QUARTER INCH BITUMINOUS MAT AT THE TIME OF CONSTRUCTION WITH THE ADDITION OF A WEARING SURFACE OF ONE INCH OR MORE IN THICKNESS AFTER THE STABILIZED GRAVEL BASE HAS BEEN UNDER TRAFFIC FOR ONE YEAR OR MORE.

Subgrade soil	Stabilized gravel base average thickness	Subgrade treatment
	In.	
Sand and Loamy Sand	2	Stabilize with binder-soil where loose.
Glacial till Sandy loam & better clay loams	4	Old roads:—correct frost boils and heaves by excavation and backfill with sand or gravel or other selected local soil.
Heavier clay loams and light clays	6 (Have built some 9 to 12)	New grading projects:—provided upper 1 to 2 ft. of roadbed be of selected soil, best available locally on the job.
Loessial silt	6 to 9	New grading projects:—provided upper 1 to 2 ft. of roadbed be of selected soil, best available locally on the job.
Lacustrine clay (highly plastic—gumbo type)	6 to 12	Generally little treatment of subgrade except to eliminate poor top-soils in forested regions.

established much as would be the case of putting down a layer of plank, so that when the next layer of aggregate or paving mixture is applied there is little or no deformation and rolling quickly produces a dense consolidated course. It is the writer's firm belief, based on observation of this procedure, that 1½ in. of screenings on a clay soil is equivalent to 3 in. of crushed stone without the use of screenings, and the inclusion of an insulation course as a requirement in the construction of new stone and gravel foundations is urged as desirable and economical construction where screenings are cheaply available.

TABLE 9

Depth of granular material added over subgrade	Cost per square yard when selected granular material ranges in cost from $0 25 to $2.00 per cubic yard				
	$0.25 cu. yd.	$0.50 cu. yd.	$1.00 cu. yd.	$1.50 cu. yd.	$2.00 cu. yd.
In.					
2	$0.02	$0.03	$0.06	$0.08	$0.11
3	.02	.04	.08	.13	.17
4	.03	.06	.11	.17	.22
5	.03	.07	.14	.21	.28
6	.04	.08	.17	.25	.33
12	.08	.17	.33	.50	.67
18	.13	.25	.50	.75	1.00

INFLUENCE OF TRAFFIC ON DESIGN

The effect of changing traffic has not yet fully been taken into account by designers. Ten years ago flexible type surfaces, particularly those of rather thin character, which had been constructed by traffic-bound methods on rural highways, were showing signs of extensive failure. Surface treatments and even penetration macadam surfaces were showing increasing costs of maintenance. This maintenance took two principal forms,—one, the repair of pot-holes where the surface was breaking down, and the other edge failure and settlement. Practically all passenger vehicles at that time were equipped with cord tires, while the majority of trucks had either solid tires or else cord tires which approached them in behavior. Impact was a decided factor because when a vehicle so equipped dropped into a depression there was little give to the tire.

This condition has completely changed and it is safe to say that the impact effect under modern tires is practically negligible. The reason is that even though a vehicle passes over a high spot in the pavement and a drop is experienced, the contact area of the balloon tire with the pavement is immediately enlarged. Recent studies on airplane tires indicate this fact to a marked degree. It explains the fact that with the general adoption of the balloon tire many old thin surfaces which were on the verge of failure in 1928 and 1929 were thereafter salvaged under simple maintenance methods and have been improving steadily ever since.

The present load per square inch under the largest truck is probably less than that under the wheels of a two-horse wagon of 20 years ago. This factor has not yet been taken into full account in pavement design, and it is noted here in order to bring out the fact that it is the *loading per square inch* and the *character of the margin of tire contact* which properly should be taken into consideration, rather than simply the total wheel

load in designing flexible types of pavement. This factor is taken into consideration by the writer in the formula suggested for determination of pavement thickness.

EFFECT OF WIDTH

The effect of width of pavement is far more important than is generally realized. Much of the failure which occurred in the late twenties was due to concentration of loads at the outer edge of the pavement. While the better ditch construction today has increased the uniformity of subgrade completely across the roadway width, nevertheless the edge of any pavement is still its weak spot and the intrusion of water beneath this edge under particularly adverse conditions of ice and snow may lead to lowered subgrade resistance. By increasing pavement width as will permit considerable lateral movement, the likelihood of any areas being over-stressed is largely eliminated.

Inasmuch as this ties up with the studies in regard to safety, it is recommended that two-lane pavement widths be a minimum of 22 ft., and for old pavements of lesser width that the berms be given such treatment as will make them practically homogeneous with the pavement itself. It is believed that the trend for two-lane pavements is toward 24 ft. as the minimum and that all experience indicates that pavements of that width can be constructed of such thickness that for a given volume of material it is better design to have this width in a thinner section than a 20 ft. width and a thicker section.

INFLUENCE OF IMPROVED EQUIPMENT

The effect of equipment applies in several ways to present day design, the first one in regard to preparation of the road cross-section. One does not have to go back very far to remember when the flow line in a highway ditch was often inadequate due to the difficulty of readily removing rock and ledge points with equipment then available. Frequent pockets would be encountered in ditches which led water to the subgrade with resultant failure of the pavement due to loss of subgrade support. This one item has been changed to such a degree through the use of power equipment that the water table under modern highways has been lowered to provide more nearly uniform subgrade support throughout the year.

Another point regarding equipment is cheap haulage. The radius of economical haul from any given source of aggregates has been greatly extended, so that it is possible to employ materials for foundation construction to take care of modern traffic needs at costs which are little more than one-half to one-third of what they were ten years ago. This coupled with easy construction of wide roadbeds and lowered water table makes for economical construction of sound foundations.

Likewise for surface courses of all types, cheap haulage has permitted mobilization of materials in storage and stockpile where they are ready with favorable weather conditions for rapid completion of the construction project in a short period. There is practically no limit to the construction rate of bituminous surface types, inasmuch as they may be used immediately after placement. Several States indicate that with cheaper hauls the type of surface has been changed. Two trends appear,—(1) that with construction of solid subgrades, heavy surface treatments are often adequate where formerly greater pavement thickness was employed, (2) that plant mixtures are becoming almost as economical as road-mixes in many areas, especially under adverse climatic conditions. Hot plant mixes are being hauled as much as 50 miles.

A complete study of the influence of equipment was presented at the Twelfth National Asphalt Conference held in Los Angeles in February, 1939, under the general chairmanship of Mr. J. L. Harrison of the Public Roads Administration. Even for those who have been constantly associated with the highway field, it is nothing short of amazing to look back and see how rapidly equipment has been improved and how greatly costs have been lowered, so that types which would not have been economical so recently as ten years ago are now invading the low cost field. A study of this report is recommended.

The improved design of bituminous mixing plants is especially notable, both respecting capacity and cost of output. The average cost of bituminous concrete has been reduced over 50 percent since 1920 and 25 percent since 1930. Some plants now turn out well over 1,000 tons per eight hour day. The development of alloy steels which permit the construction of plants having little or no maintenance and the development of the oil burners for drying have radically altered the position of plant mixtures in the paving field. A new type of "shock heat" dryer is shortly to come on the market which promises to still further extend the trend toward plant mixing of aggregates formerly placed as road mixtures.

However, with all this development in better and cheaper plants, the extension of use of bituminous mixtures is so rapid that the road-mixing types continue to gain, especially through the western and southwestern areas where adverse weather conditions are less severe. The selection of type is being made more and more as a result of study of local behavior rather than to build a type because someone else had done so. Road-mixes have a definite field of use and are particularly adapted to county and township work where tractor-grader equipment is available.

Probably in no aspect of equipment change has there been a greater improvement than in producing the final surface. One of the difficulties of flexible type construction 10 or 15 years ago was in obtaining a smooth riding surface. Old specifications are clearly evident of this fact, wherein tolerances were permitted of from $\frac{1}{4}$ in. to $\frac{1}{2}$ in. variation in 10 ft. The mechanical spreaders and more recently the combination mechanical spreaders and tampers have revolutionized this end of the work, and there is no reason today why tolerances for pavement smoothness should not be as close for flexible types of construction as for any other type. Plant mixes easily can be laid to a tolerance of $\frac{1}{8}$ in. in 10 ft. and many surfaces are being constructed under even stricter control. No plant-mix work now should be permitted (except in certain city work or small rural jobs) other than with mechanical means of spreading.

INFLUENCE OF AGGREGATE AVAILABILITY

Flexible types are different from all others in that there are a number of surfaces from which selection may be made. However in such choice, attention should be given to the availability of local aggregates, and the limitation of the manipulative process. These factors often are overlooked.

By local aggregates is meant not merely wayside products but the materials which are the output of commercial pits and quarries. Stone and gravel vary in different sections of the country in the sizes obtained from the most economical methods of reduction. Cognizance should be taken of this fact, and specifications should be written to best use the complete plant output. Such procedure is immediately reflected in cost. As an example, in Oregon where rock is plentiful, stone bases are the natural order. In Massachusetts where hard rock is found, the penetration type

of construction is a logical procedure. Where finely textured surfaces are required the employment of the penetration type as base with a thin plant-mix top is economical and employs the full quarry output. In the South the limerock and other indigenous aggregates are clearly indicated. The point to be emphasized is that design should take into account the commercial side of the picture, and that the closest cooperation should prevail between the commercial producer and the public engineer to work out the most satisfactory combinations.

THICKNESS

Closely tied in with the economical use of aggregates are the thicknesses of base and surface courses which are often governed by the character of manipulative processes. Thus in penetration macadam 2½- and 3-in. depth is best because stone can be prepared efficiently in these sizes and bituminous cement can be forced into the voids in a satisfactory manner. Bituminous concrete can be consolidated in layers up to 4 in. for coarse mixes, and to better advantage in 2- to 3-in. layers for finer mixtures. Road-mixes can be thoroughly mixed, spread and rolled in depths up to 3 in., while surface treatments can be handled best in depths less than 1 in.

Base thickness is being more and more determined by balancing cost of increased depth against cost of raising subgrade support. Usually the base thickness will be determined by the maximum amount capable of being constructed in one layer, as it is usually cheaper to increase subgrade support than to build two or more layers of base. For example, 4½-in. waterbound macadam base is an economical depth to construct in one layer and for this type such thickness is usually sufficient. It is deep enough to use commercially available stone to advantage, and to be closely keyed so that

TABLE 10

THICKNESS AND QUANTITIES FOR TYPICAL TYPES BITUMINOUS PAVEMENTS

Type	Thickness	[1]Weight of aggregate per square yard				Bituminous material per square yard	
		Coarse aggregate	Fine aggregate	Sand	Filler	Primer	Seal
	In.	*Lb.*	*Lb.*	*Lb.*	*Lb.*	*Gal.*	*Gal.*
Light surface treatment	¼ to ½	...	25	...	...	...	0.25
Heavy surface treatment	½ or more	...	30 to 60	...	...	0.5	0.3
Light road-mix seal	½ to 1	...	50 or more	...	...	0.2	0.8
Road-mix, macadam—aggregate type	2½	174	18	...	...	0.4	1.1 mix and seal
Road-mix, dense—graded type	2½	237 total	all sizes including 200 mesh	...	...		2.0 to 2.4 total
Penetration macadam surface	2½	217	35	...	...		Total 2.4
Penetration macadam base	3	260	25	...	...		Total 1.65
Bituminous concrete surface	2	126	...	65	11		Total 1.7
Bituminous concrete base	3	204	...	84	...		Total 2.1
Topeka type	2	...	59	114	20		Total 2.1
Sheet asphalt	Binder course	100	...	36	...		Total 0.85
	Top course	...	...	115	23		Total 1.9

[1]Weight aggregate based on specific gravity of 2.65.

no weak areas develop. If greater support is required it will be better usually to incorporate additional aggregate with the subgrade soil to produce mechanical stabilization and the needed support. Table 10 shows typical thickness and quantities required for various types of surface in common use.

CONCLUSIONS

1. The most important development in design of "flexible pavements" is the increasing attention to subgrade; viz., the actual design of the subgrade as a part of the pavement structure having definite minimum support values.

2. The development of the balloon tire has so reduced load concentrations that even though traffic volumes have greatly increased, the damaging effects of impact formerly experienced have practically disappeared.

3. The development of power equipment with attendant low cost of operation has greatly extended radius of economical haul of selected aggregates, and their manipulation. Surface types formerly in the high cost class are now competing in the low cost field, while the quality of all types is markedly improved.

4. In design of mixtures more attention is given to some form of stability measurement. The trend toward coarser type plant mixtures is to be noted, and that such mixtures are less critical to design in respect to stability due to the structural strength of the aggregate itself. This trend evidences itself in the newer designs which call for *thin* dense mix top and *thicker* coarse mix binder courses.

5. There is need for some simple field method of determination of subgrade support values, and correlation with a laboratory test.

6. There is need for complete study of stress distribution as related to density and stability of mixtures.

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EXPERIMENTS WITH CONTINUOUS REINFORCEMENT IN CONCRETE PAVEMENTS

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(Abstract)[1]

In the fall of 1938, the Public Roads Administration and the Indiana State Highway Commission started the construction of a number of experimental sections of reinforced concrete pavement as a part of a regular Federal Aid project. The object of these experiments was to investigate more thoroughly the possibilities of longitudinal steel reinforcement as a means for increasing slab lengths, thus decreasing the number of joints required, but at the same time achieving the control over warping made possible by the short slabs. The sections were constructed on a heavily traveled transcontinental highway west of Indianapolis, which was particularly suitable because of a fairly uniform subgrade and the absence of steep grades and sharp curves. The maximum grade is 2.95 per cent and the maximum curve is 0 deg. 11.5 min.

Three types of steel reinforcing were installed in the test sections, welded fabric, intermediate grade billet steel, and rail steel. Each slab is 10 ft. wide, and of a length to produce a range of maximum stress in the steel due to subgrade resistance as the pavement expands and contracts. For practical reasons, the standard pavement cross section used by the State of Indiana was adopted. This is a 9-7-9 thickened edge section, 20 ft. wide using a deformed metal tongue-and-groove type longitudinal joint with $\frac{5}{8}$-in. diameter tie bars spaced at 60-in. intervals.

The minimum amount of steel used corresponded to the 32-lb. welded wire fabric which is about as light as is used in pavements today. This consisted of No. 6 wire spaced at 6-in. centers longitudinally and 12-in. centers transversely. This represents about 22 lb. of longitudinal steel per 100 sq. ft. of pavement. The heaviest section of welded reinforcement was about 132 lb. of longitudinal reinforcement per 100 sq. ft. of pavement surface. The longitudinal steel was No. 4-0 wire placed 4-in. on centers, and the transverse steel was No. 3 wire, placed 12-in. on centers. The slab lengths for each of the six amounts of reinforcement used were varied so as to produce maximum stresses in the longitudinal steel of 25,000, 35,000, 45,000, and 55,000 lb. per sq. in.

Five amounts of billet steel reinforcing were used, varying from 33 to 534 lb. of longitudinal reinforcement per 100 sq. ft. of pavement surface. The lightest section consisted of $\frac{1}{4}$-in. round deformed bars set at 6-in. centers, with $\frac{1}{4}$-in. transverse bars set at 12-in. centers. The heaviest section used 1-in. round deformed bars set at 6-in. centers with $\frac{1}{2}$-in. transverse bars at 24-in. centers. Slab lengths varying from 20 to 1,080 ft. were used to produce stresses of 15,000, 25,000, 35,000, and 45,000 lb. per sq. in. in each of the five quantities of reinforcement.

Five amounts of rail steel bar rein-

[1] Presented at the Nineteenth Annual Meeting, Highway Research Board (1939), Published in full in *Public Roads*, Jan. 1940, Vol. 20, No. 11.

forcement were also used in quantities and arrangements similar to that described for the billet steel. Slab lengths were such as to stress the steel to 25,000, 35,000, 45,000, and 55,000 lb. per sq. in. for each of the five reinforcement designs.

Three types of expansion joints were used because of the wide range of section lengths. For slabs of intermediate length, a type I joint was used, which is similar to the expansion joint used at bridge approaches. Essentially, it consists of two 2-in. angles and a cover plate. The angles are anchored in opposite sides of the joint, while the cover plate is rigidly attached to one angle and held to the angle on the opposite side of the joint by a keyway system that permits sliding of the plate. The joint is designed to permit 1½-in. movement in either direction.

Type II joint is merely two of the type I joints spaced 10 ft. apart, which permits a larger affective opening, and is used where two of the longer sections are joined.

Type III joint is the conventional dowel type with ¾-in. plain round bars spaced 12 in. apart.

Both conventional and submerged type of weakened-plane joints were used. The submerged joints were placed on the subgrade so that the top of the impregnated fiber strip was uniformly 4½ in. below the top surface of the pavement. A copper seal was placed at the top of the strip to provide a water-tight seal. Dowel shear bars were used at one-half the submerged joints placed. The conventional type of surface weakened-plane joint was made with a T-bar on which was mounted a vibrator, necessitated by the fact that the concrete was placed by the vibratory method, and was quite stiff.

Four 500-ft. sections were constructed for investigating special joint designs and methods of reinforcing. Using 91- and 45-lb. welded fabric reinforcing on both submerged and surface type weakened-plane joints, the bond between the steel and concrete was broken for 18-in. on each side of the joint, and the transverse steel eliminated. The object of this design is to eliminate as far as possible, warping stresses in pavements. Breaking of the bond between steel and concrete permits warping to take place more freely, because when warping occurs, a greater length of steel is available to provide the necessary elongation without the development of excessively high stresses in the steel.

Because of the length of the project, it was necessary to let the construction in two contracts. While slightly different methods were used by the two contractors, it is felt that the effects of these differences will be negligible. Both contractors used surface vibration to ensure consolidation of the concrete, particularly in sections containing a large amount of steel. The quantity of materials used per batch on the two contracts was slightly different. One contract used crushed stone for all coarse aggregate and the other used gravel for the smaller sized coarse aggregate and crushed stone for the larger. Large sized coarse aggregate ranged from ½ to 2½ in., and the small-sized from No. 4 to 1½ in.

In constructing the sections, the paver was operated outside the forms, which allowed placing the bar mat reinforcement some distance ahead of the mixer. This type reinforcing was supported on the subgrade by chairs attached to the transverse bars by welding. The welded fabric reinforcement was placed by the strike-off method at a depth of 2½ in. from the top of the pavement. After placing the welded fabric, the top section of the pavement was poured. The concrete was cured with wet burlap until the next morning when the burlap was removed and the pavement covered with wet straw for seven days.

Because of the amount of work involved in detailed observations of the entire experimental section, it was decided that the entire pavement would be given a general condition survey once a year, and that detailed observations would be concentrated on a selected number of representative sections. These sections fall in three groups, heavily reinforced pavement, medium reinforced, and welded fabric reinforcement. The sections selected eliminate as far as possible all variables except the amount of reinforcement.

The intensive schedule of observations which is to be made on the selected sections include (1) a detailed crack survey in which a special effort will be made to find all cracks, however fine; (2) precise level measurements made on the surface of one lane of pavement; and (3) measurements of horizontal movements at the joints in the longitudinal direction of the pavement. These detailed studies are to be made three times a year, in the fall, winter and spring. Annual observations to be made over the entire length of pavement include (1) a general crack survey; (2) measurements of horizontal movements at the joints in the longitudinal direction of the pavement; and (3) general observations of the condition of the pavement.

A number of special studies are also contemplated. These include the determination of the movements of the ends of the slabs between extreme summer and winter conditions, in other words, the maximum annual change in length of various length slabs; determination of the daily movements at the ends of certain selected slabs, which will be taken on days when large temperature variations are expected and measurement of the absolute movements at the two ends, center, and quarter points of the longest slabs.

The experimental pavement was approximately one year old in October 1939. Approximately 1½ mi. had been under traffic for a period of about one year, while the remaining 4½ mi. had been in service for about six months. A large number of fine transverse cracks have occurred in the central portion of the long, heavily reinforced sections. In the long sections containing the ¾- and the 1-in. diameter longitudinal bars, the distance between the cracks is frequently less than 3 ft. There is an appreciable number of these cracks in the longer sections containing the ½-in. diameter longitudinal bars, but relatively few in the shorter sections reinforced with this amount of steel. For the sections containing the smaller amounts of reinforcement, in general the number of transverse cracks which have occurred is related more directly with the length of the sections than it is with the amount of longitudinal reinforcement. There is practically no cracking in any of the sections less than approximately 150 ft. long, regardless of the amount of reinforcement.

Typical cracks appearing in the heavily reinforced sections are not apparent except on close examination, and are very similar to those that occurred very early in the life of the heavily reinforced sections of the Columbia Pike experimental pavement.[2] Those cracks have remained closed and no serious spalling or disintegration has occurred in their vicinity. It seems unlikely, therefore, that these fine cracks in the heavily reinforced sections of this pavement will ever cause serious damage.

The sections in which the weakened-plane joints were placed at intervals of 10 ft. are in excellent condition. These sections are 500 ft. long and the steel reinforcement is continuous through the joints. Two of the sections are reinforced with a 91-lb. and two with a 45-lb.

[2] J. T. Pauls, "Reinforcing and the subgrade as Factors in the Design of Concrete Pavements," *Public Roads*, Vol. 5, No. 8.

welded fabric. The bond is broken between the steel and concrete for a distance of 36-in. at each joint.

Cracks have occurred in the surface of the pavement over all except one of the submerged weakened-plane joints. The high stressing of the steel and the breaking of the bond at the joints has allowed these joints to open an appreciable amount. The typical crack over one of these joints is quite irregular and meandering.

The weakened plane joints of the conventional type, which were formed by placing grooves in the top surface of the pavement, all appear to be in excellent condition. The manner in which the seal has been maintained at these joints is especially impressive. The tight seal can undoubtedly be attributed to the fact that the short slab lengths and the continuous steel through the joints have reduced to a very small amount the changes in width of the joints caused by the expansion and contraction of the pavement.

STRESSES IN CONCRETE RUNWAYS OF AIRPORTS

By H. M. Westergaard

Dean of the Graduate School of Engineering and Gordon McKay Professor of Civil Engineering, Harvard University

SYNOPSIS

The problem of stresses in concrete runways of airports is essentially the same as the problem of stresses in concrete pavements of highways, except that both the wheel loads that must be contemplated and the areas of contact between tire and concrete are greater for runways than for highways. These differences, especially the greater areas of contact, call for a re-examination of the formulas that have been derived for stresses in concrete pavements due to wheel loads. The study is limited to the case of loads at an appreciable distance from edges or joints.

The report of results is a report of "no surprises." The larger area of contact between the concrete pavements of the runway and the tire of each wheel of the landing gear, as compared with the corresponding areas for tires of trucks on highways, call for only very small revisions of the formulas presented by the writer previously, in the Proceedings of the Highway Research Board, 1926, and in *Public Roads*, April 1926 and December 1933. In fact, the needed revisions of the formulas are so small compared with the well-realized uncertainties, including those of the subgrade, that in most cases the corrections may be ignored. Yet the formulas for the corrections are simple and not difficult to use. In the interest of recording the evidence the fairly simple derivations of the formulas are shown at the end of the paper.

The problem of stresses in concrete runways of airports is essentially the same as the problem of stresses in concrete pavements of highways, except that the wheel loads that must be contemplated and the areas of contact between tire and concrete are greater for runways than for highways. Mr. Frank T. Sheets, President of the Portland Cement Association, has pointed out that these differences, especially the greater areas of contact, call for a re-examination of the formulas that have been derived for stresses in concrete pavements due to wheel loads. The study presented here was undertaken at the request of Mr. Sheets.

Analyses of stresses in concrete pavements due to wheel loads were presented by the writer in two articles in *Public Roads* in April 1926 and December 1933 respectively.[1] Extensive tests to examine the validity of these analyses were conducted by L. W. Teller and Earl C. Sutherland of the Division of Tests, Public Roads Administration, and were reported by them in a series of articles under the common title "The Structural Design of Concrete Pavements," *Public Roads*, October, November and December 1935, September and October 1936.

FORMULAS BASED ON THE ASSUMPTION OF A CONSTANT MODULUS OF SUBGRADE REACTION

As in the previous analyses it is expedient to begin by investigating the stresses that are created when the reaction of the subgrade per unit of area at each point is the product of a constant, the modulus of subgrade reaction k, times the deflection z of the concrete slab. The reservations that must accompany the assumption of a constant k were dis-

[1] "Stresses in Concrete Pavements Computed by Theoretical Analysis," *Public Roads*, Vol. 7, No. 2, April 1926, pp. 25-35 (also in *Proceedings*, Highway Research Board, Vol. 5, part I, pp. 90-112); "Analytical Tools for Judging Results of Structural Tests of Concrete Pavements," *Public Roads*, Vol. 14, No. 10, December 1933, pp. 185-188.

cussed at length in the two articles in *Public Roads*, April 1926 and December 1933, in which the previous analyses were presented.

Further assumptions are stated in the following list of notation:

E = modulus of elasticity of the concrete, assumed constant.

μ = Poisson's ratio of the concrete, assumed constant.

k = modulus of subgrade reaction, or reaction per unit of area per unit of deflection, measurable in lb.-in.$^{-3}$, assumed constant.

h = thickness of the concrete slab, assumed constant.

l = radius of relative stiffness, defined by equation (1).

P = total pressure exerted by one wheel, assumed to be applied at a considerable distance from any edge of the slab or joint in the slab.

a = radius of a circle over the area of which P is assumed to be distributed uniformly.

b = radius depending on a and h, defined by equations (2) and (3).

σ = tensile stress at the bottom of the slab directly under the center of the wheel pressure.

σ_1 = value computed for σ by the formulas which were derived in the previous analysis under the assumption of only small values of the radius a.

σ_2 = supplementary stress which must be added to σ_1 to give the proper stress σ; σ_2 will be significant only for larger values of the radius a.

z_0 = deflection of the slab at the center of the load.

z_1 = value of z_0 when a is very small.

The following formulas are quoted from the article in *Public Roads*, December 1933:

$$l^4=\frac{Eh^3}{12(1-\mu^2)k} \tag{1}$$

$$b=\sqrt{1.6a^2+h^2}-0.675h \quad \text{when } a<1.724h \tag{2}$$

$$b=a \text{ when } a>1.724h \tag{3}$$

$$\sigma_1=0.275(1+\mu)\frac{P}{h^2}\log_{10}\left(\frac{Eh^3}{kb^4}\right) \tag{4}$$

$$z_1=\frac{P}{8kl^2} \tag{5}$$

Equations (1), (2), (3) and (5), and the equivalent of equation (4) for $E=3{,}000{,}000$ lb. in.$^{-2}$ and $\mu=0.15$, were also stated in the article in *Public Roads*, April 1926.

The present study has given the following results, satisfactorily applicable when a does not exceed l:

(a) The supplementary stress to be added to σ_1 is approximately

$$\sigma_2=\frac{3(1+\mu)P}{64h^2}\left(\frac{a}{l}\right)^2 \tag{6}$$

The result in equation (6) was also derived independently and by a different process of analysis by W. D. Dickinson, Jr., who made a study of the problem at the request of the writer.

(b) Since $1-\mu^2$ varies only slowly with μ, it is justifiable to substitute $\mu=0.15$ in equation (1) for the purpose of computing stresses. This substitution permits the following restatement of equation (6):

$$\sigma_2=\frac{0.16(1+\mu)Pa^2}{h^3}\sqrt{\frac{k}{Eh}} \tag{7}$$

(c) The resultant stress

$$\sigma=\sigma_1+\sigma_2 \tag{8}$$

is the greatest tensile stress produced by the load P. The radius a would have to be considerably greater than l to cause the maximum tensile stress to occur at any other point than the center of the circle.

(d) The restriction

$$a<l \tag{9}$$

is found to be approximately equivalent to the condition

$$\sigma_2 < 0.16\sigma_1 \qquad (10)$$

Condition (10) may be used instead of (9) in deciding whether the formulas are within the range of applicability.

(e) The deflection under the center of the load is approximately

$$z_0 = \frac{P}{8kl^2}\left[1 + \left(0.3665 \log_{10}\left(\frac{a}{l}\right) - 0.2174\right)\left(\frac{a}{l}\right)^2\right] \qquad (11)$$

The derivation of equations (4), (6) and (11) is described in the last section of this paper.

NUMERICAL EXAMPLE

To illustrate the use of the formulas the possibility of using a 6-in. slab is investigated. Assume

$$E = 4{,}000{,}000 \text{ lb. in.}^{-2}, \quad \mu = 0.15, \quad k = 100 \text{ lb. in.}^{-3} \qquad (12)$$

$$h = 6 \text{ in.}, \quad P = 30{,}000 \text{ lb.}, \quad a = 13.8 \text{ in.} \qquad (13)$$

The value of a corresponds to a pressure of 50 lb. in.$^{-2}$ over the loaded area. Equations (3), (4), (7), and (8) give

$$\sigma_1 = 626 \text{ lb. in.}^{-2}, \quad \sigma_2 = 10 \text{ lb. in.}^{-2}, \quad \sigma = 636 \text{ lb. in.}^{-2} \qquad (14)$$

Since σ_2 is less than 16 per cent of σ_1, the formulas are within the range of applicability, which is also verified by computing l from equation (1) and noting that a is less than l. Equations (1), (5), and (11) give

$$l^2 = 858 \text{ in.}^2, \quad l = 29.3 \text{ in.} \qquad (15)$$

$$z_1 = 0.0437 \text{ in.}, \quad z_0 = 0.0404 \text{ in.} \qquad (16)$$

$$\frac{z_0}{z_1} = 0.925 \qquad (17)$$

FORMULA BASED ON COEFFICIENT OF SUBGRADE STIFFNESS

In the article in *Public Roads*, December 1933, it was proposed to introduce a "coefficient of subgrade stiffness," K, defined as

$$K = kl \qquad (18)$$

K is measurable in pounds per square inch and may be less dependent on the properties of the slab than k is likely to be. The following formula is practically a restatement of equation (4) and is quoted from the article in *Public Roads*, December 1933:

$$\sigma_1 = \frac{1.1(1+\mu)P}{h^2}\left[\log_{10}\left(\frac{h}{b}\right) + \tfrac{1}{3}\log_{10}\left(\frac{E}{K}\right) - 0.089\right] \qquad (19)$$

With the values in the preceding numerical example one finds

$$K = 2940 \text{ lb. in.}^{-2} \qquad (20)$$

and σ_1, σ_2 and σ unchanged.

SUPPLEMENTARY COMPUTATION CONTEMPLATING A REDISTRIBUTION OF THE SUBGRADE REACTIONS

The preceding computations are based on the assumption that the reactions of the subgrade are distributed according to a definite value of the modulus k or the coefficient K. Actually the reactions must be expected to have a different distribution. It is desirable, therefore, to consider the effects of a plausible redistribution of the reactions of the subgrade. The last two sections in the article in *Public Roads*, December 1933, deal with this subject. The following formula for a supplementary stress, to be superposed on σ_1 (or σ) is taken from that article:

$$\sigma_3 = -\frac{15(1+\mu)CP}{h^2}\left(\frac{l}{L}\right)^2 \qquad (21)$$

in which L is a distance and C (denoted Z in the article referred to) is the ratio of reduction of z_1 due to the redistribution of the reactions.

An extreme case is of particular interest; namely, that in which the subgrade behaves as a deep elastic solid with constant modulus of elasticity in compres-

sion, E_s, and a constant Poisson's ratio μ_s. The following relations were found to represent this case when a is small.

$$L=5l, \quad C=0.3906,$$
$$K=0.1242\frac{E_s}{1-\mu_s^2} \tag{22}$$

The present study contemplates larger values of a. It appears to be a plausible estimate that C then should be reduced to the value

$$C=0.3906\frac{z_0}{z_1} \tag{23}$$

With the values in the preceding numerical example equations (23) and (17) give

$$C=(0.3906)(0.925)=0.3613 \tag{24}$$

Then equation (21) gives

$$\sigma_3=-\frac{15(1+0.15)(0.3613)(30{,}000\text{ lb.})}{6^2\text{ in.}^2}\left(\frac{1}{8}\right)^2=-208\text{ lb. in.}^{-2} \tag{25}$$

so that the resultant stress would be

$$\sigma'=\sigma_1+\sigma_2+\sigma_3=(626+10-208)\text{ lb. in.}^{-2}$$
$$=428\text{ lb. in.}^{-2} \tag{26}$$

The actual resultant stress may be expected to lie between the extremes $\sigma=636$ lb. in.$^{-2}$ and $\sigma'=428$ lb. in.$^{-2}$.

LIMITATION OF THE ANALYSIS

It should be kept in mind that the computations presented here deal only with the case in which the load is at an appreciable distance from any edge of the slab or joint. Loads at a joint call for further investigation.

DERIVATION OF THE BASIC FORMULAS

It remains to show the derivation of the basic formulas, (4) and (6) for the stresses and (11) for the deflections.

Let

r = horizontal radial distance from the center of the load, and
z = deflection at any point.

Since z will be a function of r only, Laplace's operator assumes the form

$$\Delta=\frac{d^2}{dr^2}+\frac{1}{r}\frac{d}{dr} \tag{27}$$

Any unloaded part of the slab must satisfy the equation [2]

$$\frac{Eh^3}{12(1-\mu^2)}\Delta^2 z+kz=0 \tag{28}$$

By introducing l according to equation (1), equation (28) assumes the convenient form

$$l^4\Delta^2 z+z=0 \tag{29}$$

Now consider the function

$$Z=z+il^2\Delta z \tag{30}$$

in which $i=\sqrt{-1}$. It is observed that

$$l^2\Delta Z+iZ=i(l^4\Delta^2 z+z) \tag{31}$$

It follows that equation (29) is satisfied if Z is chosen as a function of r satisfying the equation

$$l^2\Delta Z+iZ=0 \tag{32}$$

and z is taken as the real part of Z. According to equation (30) we may write

$$z=\text{Re}Z, \quad \Delta z=\frac{1}{l^2}\text{Im}Z \tag{33}$$

the symbols Re and Im standing for "the real part of" and "the imaginary part of," respectively.

Equation (32) is satisfied by any Bessel function of order zero with argument $\frac{r\sqrt{i}}{l}$. The Bessel function of interest here is "Hankel's Bessel function" $H_0^{(1)}$. Information concerning this function is given in convenient form in the book of tables by Jahnke and Emde.[3]

[2] See, for example, *Public Roads*, March 1930, p. 4; or, A. Nadai, "Die elastischen Platten" (Julius Springer, Berlin), 1925, especially p. 186.

[3] E. Jahnke and F. Emde, "Funktionentafeln mit Formeln und Kurven" (B. G. Teubner, Leipzig), 1909, especially pp. 96 and 139.

Since Hankel's Bessel function $H_0^{(2)}$ will not be used here, the top index (1) will be omitted in the equations that follow. The function [4]

$$Z=\frac{1}{4kl^2}H_0\left(\frac{r\sqrt{i}}{l}\right) \qquad (34)$$

not only satisfies equation (32), but converges toward zero at infinity, and can be shown to have the proper singularity at the origin to account for a concentrated load $P=1$ at that point. For,[5]

$$\frac{dz}{dr}=\frac{d}{dr}(\mathrm{Re}Z)=0 \quad \text{at} \quad r=0 \qquad (35)$$

and for small values of r

$$l^2\Delta z=\mathrm{Im}Z=\frac{1}{4kl^2}\frac{2}{\pi}\log_e\frac{\gamma r}{2l} \qquad (36)$$

in which γ is a constant. On a circular section defined by a small constant value of r the vertical shear per unit of length may be computed, by referring to equation (1), as [6]

$$v=-kl^4\frac{\partial\Delta z}{\partial r}=-\frac{1}{2\pi r} \qquad (37)$$

making the total shear -1 as it should be.

A load 1 distributed uniformly over the circumference of a circle defined by a constant value of r produces the same values of the deflection z and the curvature Δz at the origin as a load 1 at the origin produces at the distance r from the origin. Consequently, the values z_0 of z and $(\Delta z)_0$ of Δz produced at the origin by a load P distributed uniformly over the area of the circle $r=a$ may be determined by forming the integral

$$W=\int_0^a Z\cdot\frac{P}{\pi a^2}\cdot 2\pi r dr \qquad (38)$$

and computing

$$z_0=\mathrm{Re}W \text{ and } (\Delta z)_0=l^{-2}\,\mathrm{Im}W \qquad (39)$$

The curvature $(\Delta z)_0$ at the origin defines the bending moment m_0 at the origin per unit of length of section by the formula.

$$m=-\frac{1+\mu}{2}kl^4(\Delta z)_0 \qquad (40)$$

This bending moment defines the tensile stress at the bottom at $r=0$:

$$\sigma=\frac{6m_0}{h^2} \qquad (41)$$

Consequently

$$\sigma=-\frac{3(1+\mu)kl^2}{h^2}\,\mathrm{Im}W \qquad (42)$$

The next step is to express W. One finds by equations (34) and (38)

$$W=\frac{P}{2kl^2a^2}\int_0^a r dr\, H_0\left(\frac{r\sqrt{i}}{l}\right) \qquad (43)$$

or

$$W=\frac{P}{2ika^2}\int_0^{\frac{a\sqrt{i}}{l}}\frac{r\sqrt{i}}{l}d\left(\frac{r\sqrt{i}}{l}\right)H_0\left(\frac{r\sqrt{i}}{l}\right) \qquad (44)$$

The integral in equation (44) can be expressed in terms of Hankel's Bessel function H_1 of order one, as follows: [7]

$$W=\frac{P}{2ika^2}\left[\frac{a\sqrt{i}}{l}H_1\left(\frac{a\sqrt{i}}{l}\right)+\frac{2i}{\pi}\right] \qquad (45)$$

which may be restated as

$$W=\frac{P}{2kl^2}\left[\frac{1-i}{\sqrt{2}}\frac{l}{a}(\mathrm{Re}+i\mathrm{Im})\,H_1\left(\frac{a\sqrt{i}}{l}\right)+\frac{2}{\pi}\left(\frac{l}{a}\right)^2\right] \qquad (46)$$

[4] Compare the footnote on page 34 in the article in *Public Roads*, April 1926, which contains references to earlier investigations. Further information concerning the technique of analysis of problems of this type is found in the book by F. Schleicher, "Kreisplatten auf elastischer Unterlage" (Julius Springer, Berlin), 1926, 148 pp., which contains extensive analyses of problems of circular slabs on elastic support.

[5] Jahnke and Emde, *loc. cit.*, pp. 97 and 141.

[6] See for example, *Public Roads*, March 1930, p. 4, where in equations (16), by a misprint, the symbol ∂ is omitted at two places. The first equation should be

$$V_x=-N\frac{\partial\Delta z}{\partial x}$$

[7] Jahnke and Emde, *loc. cit.*, pp. 166 and 98.

By referring to equations (39), (42) and (46) one finds the deflection at the center of the loaded area

$$z_0=\frac{P}{2kl^2}\left[\frac{l}{a\sqrt{2}}\,(\mathrm{Re}+\mathrm{Im})\,H_1\left(\frac{a\sqrt{i}}{l}\right)+\frac{2}{\pi}\left(\frac{l}{a}\right)^2\right] \quad (47)$$

and the tensile stress at the bottom of the slab under the center of the load

$$\sigma=\frac{3(1+\mu)P}{2h^2}\,\frac{l}{a\sqrt{2}}\,(\mathrm{Re}-\mathrm{Im})\,H_1\left(\frac{a\sqrt{i}}{l}\right) \quad (48)$$

Equations (47) and (48) apply regardless of the size of the loaded circular area, except that when the radius a is small, the value a should be replaced by b defined in equation (2), for reasons explained in the article in *Public Roads*, April 1926.

The function $H_1\left(\frac{a\sqrt{i}}{l}\right)$ can be expressed[8] as the sum of a convergent series of uneven powers of $\frac{a}{l}$ plus the factor $\log_e\frac{\gamma a}{2l}$ times another convergent series of uneven powers of $\frac{a}{l}$; the number γ being such that

$$\log_e\left(\frac{\gamma}{2}\right)=-0.1159 \quad (49)$$

Consequently z_0 and σ in equations (47) and (48) can be expressed similarly by means of series of even powers of $\frac{a}{l}$. These series converge so well that for $a<l$ very good approximations are found by including only the terms of power zero and two. Thereby the following formulas are found:

$$\sigma=\frac{3(1+\mu)P}{2\pi h^2}\left[\log_e\frac{2l}{\gamma a}+\tfrac{1}{2}+\frac{\pi}{32}\left(\frac{a}{l}\right)^2\right] \quad (50)$$

$$z_0=\frac{P}{8kl^2}\left[1+\frac{1}{2\pi}\left(\log_e\frac{\gamma a}{2l}-\frac{5}{4}\right)\left(\frac{a}{l}\right)^2\right] \quad (51)$$

By use of equation (49) equations (50) and (51) are converted readily into equations (4), (6), (8) and (11).

That the bending moments have their maximum value at the center of the loaded circle when a does not exceed l, is ascertained by an examination of the moment diagrams shown in the article in *Public Roads*, April 1926. By interpreting these moment diagrams as influence diagrams it is observed that when a does not exceed l, a small movement of the loaded circular area will always reduce the moments at the original center.

DISCUSSION ON STRESSES IN CONCRETE RUNWAYS

DR. D. L. HOLL, *Iowa State College:* If we were to examine some of the elements in the formula presented, our questions would be with reference to a matter of certain judgments that enter into the expressions involved. In granting that all of the assumptions employed in the original solution are valid, we come to the place where a disturbing infinite stress value arises due to a point load. Here the author introduces another radius known as the equivalent radius of loading b. Am I misinformed that this radius b was derived for slabs having no distributed support but is utilized here with a reaction load underneath the slab?

DR. H. M. WESTERGAARD, *Harvard University:* Yes, that is quite true. The reaction can be interpreted as a distributed load and the behavior of the slab under a distributed load is reasonably well in ac-

[8] Jahnke and Emde, *loc. cit.*, pp. 97 and 141; numerical table on p. 140.

cordance with the ordinary theory of reflexive slabs so therefore whatever equivalent radius you find for a slab separated at the edge should also be reasonably well applicable to a slab that has distributed support; but that is not so much the point in the present paper. Here we are dealing with large areas of contact; in other words, a load is farther from being a bound load than in the case of a highway and there is no necessity for making a correction.

DR. HOLL: In the introduction of supplementary stresses, it appears to me that the paper would be considerably improved if we could have some rational basis for the judgment factors that enter such as the reduction constant "C," the factor 15 and the radius "L," which are to be employed by the designer in his flexural stress calculations; for example I would not know whether I should use $\left(\frac{l}{L}\right)$ to be $\frac{1}{2}$, $\frac{1}{8}$ or $\frac{1}{10}$, in which case the square of such a small fraction could disturb the equation considerably, or why should we choose that factor $\left(\frac{l}{L}\right)$ and then determine the range within which the reduction constant "C" is employed? I think it would greatly enlarge the interest and usefulness of the paper if we knew these bases.

DR. WESTERGAARD: I refer you to *Public Roads* December 1933 in which I think a rational reason is given for certain values. The reason is given in that earlier paper. The present paper would have been too long if I had put it in again.

MR. R. D. BRADBURY, *Wire Reinforcement Institute:* It so happens that the factor which Dr. Holl questions was not involved in Dr. Westergaard's paper, since the areas of contact applied by the landing wheels of airplanes are so large that the radius of contact area may be used instead of the so called equivalent radius "b" as is used in analyzing stresses in concrete roads.

PROFESSOR M. G. SPANGLER, *Iowa State College:* The formula which Dr. Westergaard propounded a number of years ago for a corner load situation, indicated that the locus of maximum stress produced by a corner load was some distance back from the corner. Later on, experiments conducted at Ames, in which stresses in the top surface of the pavement slab were measured, verified that conclusion. Also, if my memory serves me correctly, the experiments conducted by the Public Roads Administration did likewise. But it has been noticed that a lot of the pavement corner breaks which develop under traffic do not occur at an appreciable distance back from the corner, but seem to begin fairly close to the corner and the failure progresses toward the interior of the slab until a considerable area adjacent to the corner is affected. Some engineers have called this type of failure, "Map Cracking" although this term is not limited to the cracking in the corner region to which I have reference, and which is illustrated in Figure 1. I have been very much interested in this phenomenon, wondering why many of these corner failures begin so close to the corner rather than at some appreciable distance back from the corner as indicated by both theory and experiment.

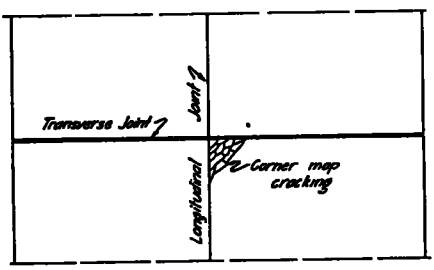


Figure 1. Corner Map Cracking of Concrete Pavements

I do not pretend to know the answer to this problem, but would like at this time simply to suggest that perhaps we have not been taking into account all the forces to which a pavement is subjected when a wheel load passes over a corner of the slab, in the development of a stress theory and in the conduct of our experiments. In addition to the vertical gravity loads, both static and impact, which are taken into account in corner load stress calculation, there are certain horizontal or surface shear loads which may exist at the area of contact between a tire and the pavement which are not ordinarily taken into account. These tangential forces may be due to the tractive effort or resistance between the tire and the pavement, and, probably to a much less extent, to stresses accompanying the flexure of a tire as it grips the pavement.

The tangential forces due to tractive effort between a driving wheel and a pavement may be quite high. As a maximum situation, consider a wheel load of 8,000 lb. operating on a dry, rough, concrete surface. Under these conditions, the coefficient of friction between tire and pavement may be as high as 0.7 or 0.8, and if the vehicle is being accelerated rapidly enough, the tangential force may be roughly 5,600 to 6,400 lb. This represents a maximum possible situation and not necessarily a probable case. The direction of this force, when the vehicle is accelerating, will be opposite to the direction of travel. From this possible maximum, the tangential force may range downward through zero, when the vehicle is rolling along without acceleration or deceleration, to a maximum force of equal magnitude in the opposite direction when the vehicle is decelerating or braking at its maximum rate.

Now these tangential forces will produce either tensile or compressive stresses in the top surface of the slab, depending upon their direction, which will combine with the tensile bending stress produced by the vertical load. It has been shown experimentally that the bending stresses due to vertical loads change very slowly along the corner bisector and they may be nearly as great right near the load as at the locus of maximum stress at some distance away from the load, as shown in Figure 2. Therefore, in the immediate

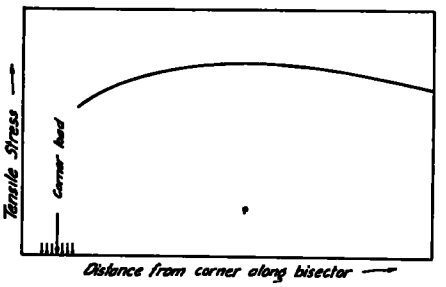


Figure 2. Variation in Bending Stress Caused by Vertical Corner Load

vicinity of the load, or even within the contact area, the stresses due to the tangential force may combine with the bending stresses due to the vertical load and reach a maximum combined stress which is greater than maximum stress due to vertical loads alone. These combined stresses may be responsible for the occurrence of "map cracking" in a corner region.

MR. BRADBURY: I have often noticed the condition to which Mr. Spangler refers. It seems to me that these breaks, which occur only a few inches from the corner, might be more a case of aggravated spalling than a true flexural corner break. If they were structural breaks, they would be expected to occur 2 to 3 feet from the corner.

MR. E. C. SUTHERLAND, *Public Roads Administration:* The Public Roads Administration has made certain studies of the stress conditions occurring in the vicinity of interior corners of concrete pavements. This study included both the

stresses caused by loads acting near the corners and those caused by restrained temperature warping. It was found that the longitudinal and transverse joint connections between the four interior corners cause that part of the pavement in the vicinity of the interior corners to act in a manner similar to that of the interior of a pavement slab. That is, a load placed near an interior corner frequently causes a critical stress condition directly under the load while the stresses occurring along the bisector of the corner angle, which are normally high on free corners, may be low at interior corners. This is shown by a specific test in which loads were applied at both the free and interior corners of a pavement slab and the stresses measured directly under the loads and along the bisector of the corner angles. At the free corners the stress directly under the load was so small that it was not measurable while the maximum stress on the bisector of the corner angle, some distance from the load, was 321 pounds per square inch. At the interior corners the average stress directly under the load was 272 pounds per square inch, while the maximum stress on the bisector of the corner angle, some distance from the load, was 84 pounds per square inch.

More complete data, on this subject, were presented in the reports of the Arlington test pavement.[1] In this report the following statement is made concerning the stress conditions at interior corners of concrete pavement slabs.

"It is interesting to note that at the inside corners, where the load stresses along the bisector of the corner angle are very low, the stresses directly under the load become relatively high. This is due to the action of the joints causing the slab at point D to behave more in the manner of the interior of the slab. One joint acting effectively will cause the stresses at this point to be distributed as at a free edge, while with both joints effective a stress distribution more like that which exists in the case of an interior loading is created. Thus the position and magnitude of the critical stress at a slab corner depend upon the action of the joint or joints at that corner. Joints that are very effective in controlling the stress along the bisector of the corner angle may cause a critical stress condition under a load acting near a corner.

"It has already been shown that from the standpoint of reducing warping stresses, free action of the corners at point 'D' is desirable. Such construction would likewise reduce the load stress just discussed and increase slightly the load stress along the bisector of the corner angle of the slab. Therefore, as far as both warping and load stresses are concerned, the joints should be so designed that resisting moments that prevent free flexure are not developed in the joint."

[1] The Structural Design of Concrete Pavements by L. W. Teller and Earl C. Sutherland, Part 4, *Public Roads*, Vol. 17, Nos. 7 and 8, September and October 1936, Pages 185-6.

PROGRESS IN STUDY OF MOTOR VEHICLE PASSING PRACTICES

By O. K. Normann

Associate Highway Economist, Public Roads Administration

SYNOPSIS

At the eighteenth annual meeting of the Highway Research Board in 1938, Mr. E. H. Holmes of the U. S. Public Roads Administration presented a paper describing the equipment and procedure employed in passing studies. At that time the equipment had been used on four study sections in Maryland and Virginia. During the past summer, studies have been conducted in cooperation with State officials in Massachusetts, Illinois, Ohio and Texas.

The detailed data for 1,635 passings from the studies in Maryland and Virginia show that 33 per cent of the maneuvers were multiple passings involving a total of 57.3 per cent of the passings that took place with an average hourly volume of 375 vehicles. This fact indicates the importance of studying multiple passing maneuvers as well as maneuvers in which one vehicle passes only one vehicle. Of the multiple passing maneuvers, 63.5 per cent were composed of maneuvers in which one vehicle passed, or was passed, by two vehicles. A very definite relationship was found to exist between the traffic volume and the number of passings per hour, and also between the traffic volume and the percentage of the total maneuvers that were multiple passings.

In nearly 85 per cent of the single passing maneuvers that took place when the traffic volume averaged 375 vehicles per hour, the passing vehicles slowed down to some extent before attempting to pass, and in 53.7 per cent the passing vehicle slowed down to the same speed as the vehicle to be passed.

Although a comprehensive analysis of the passings has just begun, indications are that the data derived from these studies will yield even more information relative to driver psychology and the design of highways for safe and uncongested movement of traffic than had been anticipated.

Knowledge of the manner in which highways are used is a prerequisite to improving their design so that they will more adequately serve highway users. A study of passing practices of motor vehicles is part of a research program recently initiated by the Public Roads Administration to supply information on the normal driving habits of vehicle operators.

During the fall of 1938 studies of the passing practices of motor vehicles were conducted on four sections of highway in Maryland and Virginia with special equipment developed by the Public Roads Administration. A report describing in detail the methods and equipment used, and the purposes of the passing practice studies, has been published.[1]

[1] Procedure Employed in Analyzing Passing Practices of Motor Vehicles, by E. H. Holmes, *Public Roads*, Vol. 19, No. 11, January 1939, *Proceedings*, Highway Research Board, Vol. 18, p. 368.

In cooperation with State highway officials, studies were conducted during the summer of 1939 in Massachusetts, Ohio, and Illinois, and studies are now being conducted in Texas. The program also includes studies in California and Oregon next spring. Upon completion of the field work, data will be available for normal passing practices under a wide variety of road conditions, geographically distributed to include any major differences in driving habits.

Several improvements have been made in the equipment to reduce the time required for installation on the study sections and to permit operation at night and on rainy days. The most important improvement has reduced the amount of work required in transcribing the field records to less than two-thirds of its former cost.

The detailed data for 1,635 passing maneuvers recorded during 37½ hours of

operation on the four study sections in Maryland and Virginia are now ready to be placed on tabulating machine cards prior to starting the comprehensive analysis.

Although these 1,635 passings are but a small portion of the total number that will be obtained during this study, they illustrate the method of analysis and some of the facts with respect to passing practices and driver behavior that are being obtained.

passing distances and practices a study of multiple passing maneuvers as well as single passing maneuvers.

The most important multiple passing maneuvers are those in which one vehicle either passes or is passed by two vehicles. They compose 63.5 percent of the multiple maneuvers or 46.6 percent of the passings accomplished by multiple maneuvers. Three vehicles passing four other vehicles was the most complicated multiple passing maneuver recorded.

TABLE 1

TYPES OF PASSING MANEUVERS OBSERVED

(Average volume 375 vehicles per hour)

Type of maneuver	Maneuvers made		Passings accomplished	
	Number	*Percent*	*Number*	*Percent*
Single	1,096	67.0	1,096	42.7
Multiple				
1 vehicle passing 2 vehicles	181	11.1	362	14.1
2 vehicles passing 1 vehicle	161	9.8	322	12.6
1 vehicle passing 3 vehicles	63	3.9	189	7.4
2 vehicles passing 2 vehicles	42	2.6	168	6.5
3 vehicles passing 1 vehicle	30	1.8	90	3.5
1 vehicle passing 4 to 6 vehicles	31	1.9	136	5.3
2 vehicles passing 3 to 5 vehicles	13	0.8	102	4.0
All other multiple passings	18	1.1	99	3.9
Total multiple	539	33.0	1,468	57.3
Grand total	1,635	100.0	2,564	100.0

The first classification of the passing maneuvers was made by separating them into the single and multiple passing types. In the single passing maneuvers, one vehicle passed one other vehicle, while in the multiple passing maneuvers, two or more vehicles either passed or were passed by one or more vehicles.

Table 1 shows that 57.3 percent of the passings were accomplished by multiple maneuvers although there were only about half as many multiple maneuvers as there were single maneuvers (one passing two other vehicles accounts for two passings). These figures illustrate the importance of including in a study of

Figure 1 shows, for various hourly volumes, the percentage of the total number of maneuvers and passings ac-

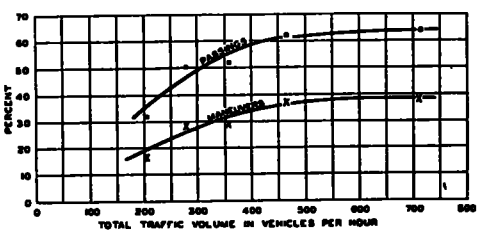


Figure 1. Percentage of Total Passing Maneuvers and Total Number of Passings That Were Accomplished by Multiple Passing Maneuvers at Various Traffic Volumes.

complished by multiple passing maneuvers. At an hourly traffic volume of 200 vehicles, 35 percent of the total passings were accomplished by multiple maneuvers. At traffic volumes above 450 vehicles per hour this figure exceeds 60 percent.

Figure 2 shows the average number of maneuvers and passings observed per hour on the four half-mile study sections during various hourly traffic volumes. As expected, there is a marked increase in the number of passings as the volume increases. These are the number of passings accomplished per hour and not the number of passings that would have been made had all vehicles that desired to pass been able to make passing maneuvers.

One vehicle usually passes another because the driver wants to travel faster than the other vehicle is moving. Within the half-mile study section it was generally possible to determine the speed that the driver of the passing vehicle desired to travel by noting his speed either before slowing down prior to making the passing maneuver or after the maneuver was completed.

TABLE 2

SINGLE PASSINGS CLASSIFIED BY THE SPEED OF THE PASSED VEHICLE AND THE DESIRED SPEED OF THE PASSING VEHICLE

Desired speed of passing vehicle in miles per hour faster than speed of passed vehicle	Speed of passed vehicle in miles per hour					
	20 and under	21-30	31-40	41-50	Over 50	Total
	Percent	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
5 and under	...	1.9	11.2	7.8	0.3	21.2
6-10	...	4.0	18.8	7.1	0.3	30.2
11-15	0.4	6.7	17.6	5.5	0.3	30.5
16-20	0.7	5.0	5.7	0.8	...	12.2
21-30	0.3	2.9	1.6	0.3	0.1	5.2
Over 30	0.3	0.2	0.1	0.1	...	0.7
Total	1.7	20.7	55 0	21.6	1.0	100.0
Average difference is speed between passed and passing vehicle (m.p.h.)						
	20.6	14.2	10.5	8.6	11.1	10.9

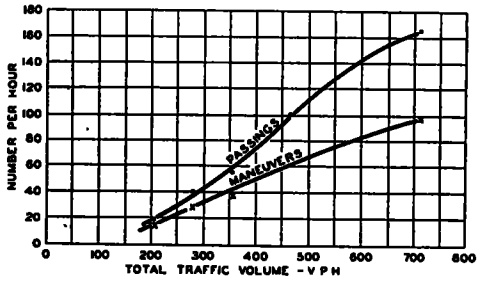


Figure 2. Total Number of Maneuvers and Passings Accomplished per Hour on one-half Mile Study Sections at Various Hourly Traffic Volumes.

Table 2 shows that in 55 percent of the passings the passed vehicle was traveling from 31 to 40 miles per hour. The speeds of the passed vehicles in the remaining passings were almost equally distributed between the 21 through 30 and 41 through 50 mile-per-hour groups.

This table also shows that 51.4 percent of the drivers that passed desired to travel less than 11 miles per hour faster than the passed vehicle and that the desired speed of 21.2 percent was less than 6 miles per hour faster. There is a

marked decrease in the average difference between the speed of the passed vehicle and the desired speed of the passing vehicle as the speed of the passed vehicle increases. Besides showing the frequency distribution of the speeds of passed vehicles, these data indicate that drivers desiring to travel at a slightly higher speed than the vehicle ahead would rather pass the preceding vehicle when the opportunity presents itself than reduce their speed slightly and stay behind.

Of all the drivers that were able to accomplish single passing maneuvers on the study sections, Table 3 shows that 84.4 percent had to slow down before they could start to pass; 53.7 percent slowed down to practically the same speed as the vehicle they were going to pass and 16 percent slowed down to within 5 miles per hour of the speed of the vehicle they were going to pass. About one third had to slow down and stay behind the preceding vehicle until they could see that the road was clear for a sufficient distance ahead to permit them to pass, and 50.9 percent had to slow down and wait for an oncoming vehicle to pass before they could start the passing maneuver. The other 15.6 percent were not required to slow down prior to starting the maneuver. They may have had to slow down after completing the maneuver but they started the maneuver at their normal speed.

When the drivers of the passing vehicles had completed the passing maneuvers and returned to the right-hand lane, Table 4 shows that the left lane was clear for a distance of less than 500 ft. in 27 percent of the passings and that there was an oncoming vehicle less than 500 ft. away in 16.8 percent of the passings.

TABLE 3

PERCENTAGE OF VEHICLES MAKING SINGLE PASSING MANEUVERS THAT WERE DELAYED BEFORE STARTING TO PASS

(Average traffic volume 375 vehicles per hour)

	Delayed by insufficient sight distance	Delayed by oncoming vehicle	Total
	Percent	*Percent*	*Percent*
Slowed down to same speed as vehicle to be passed	18.0	35.7	53.7
Slowed down to within 5 m.p.h. of the speed of the vehicle to be passed	6.6	9.4	16 0
Slowed down, but not to within 5 m.p.h. of speed of vehicle to be passed	8.9	5.8	14.7
Total delayed in starting maneuver	33.5	50.9	84.4
Total not delayed in starting maneuver	....	....	15.6
			100.0

TABLE 4

DISTANCE THAT DRIVERS OF PASSING VEHICLES COULD SEE THAT LEFT LANE WAS CLEAR AT THE TIME THE PASSING MANEUVER WAS COMPLETED

Distance that left lane was clear	Sight distance limiting factor	Oncoming car in view	Total
Feet	*Percentage of total maneuvers*		
Less than 500	10.2	16.8	27.0
500 to 1,000	19.0	16.0	35.0
Over 1,000	30.5	7.5	38.0
Total	59.7	40.3	100.0

The data for the passings in which the passing vehicle was not forced back into the right lane may not be very useful in determining minimum passing distances but they will show actual driving practices during unrestricted conditions. Driving practices during unrestricted as well as restricted conditions must be known when designing highways to fit the normal driving habits of vehicle operators.

The data obtained for each passing maneuver are illustrated by Figures 3 to 9 inclusive, each representing one of seven critical positions. All speeds, distances, time intervals and relative positions of each vehicle with respect to the other vehicles as shown in these figures were obtained from the data sheet for one passing maneuver.

Similar data have been compiled for the 1,635 maneuvers recorded on the study sections in Maryland and Virginia and also for 500 of the maneuvers recorded during the Massachusetts studies. It is not intended that this much detail be obtained for all the thousands of passings that will have been recorded when the scheduled field work is completed, but the factors that appear to be the most important as the analysis progresses will be taken from the field records for enough maneuvers to obtain a representative sample of each type of maneuver at a series of speeds for each available road condition.

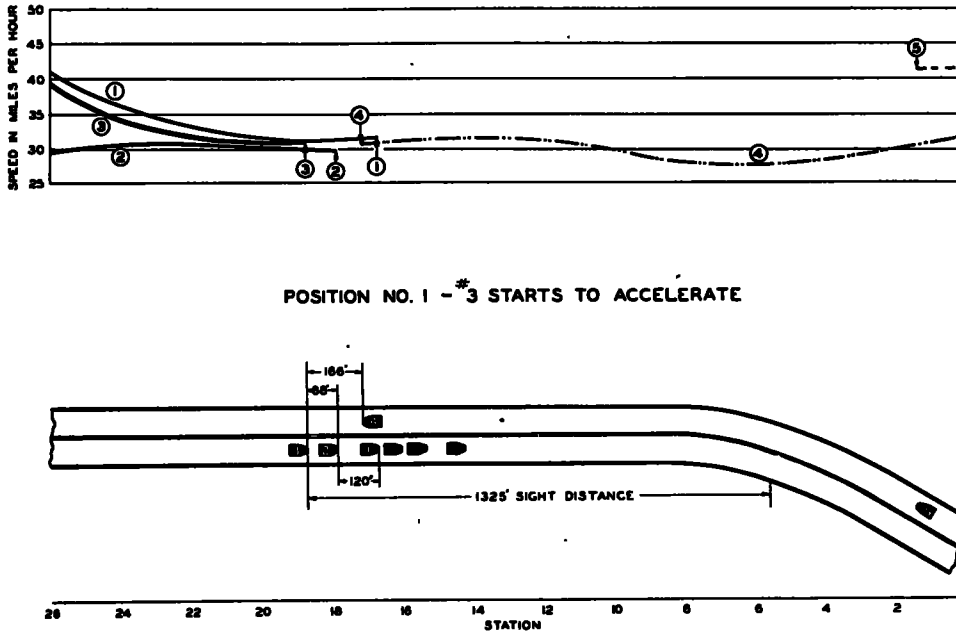


Figure 3

Figure 3 shows the position of each vehicle that is likely to affect the manner in which the passing maneuver is made. At this instant, vehicle No. 3 starts to accelerate in order to pass vehicle No. 2 and possibly the four vehicles ahead of vehicle No. 2, the closest one being vehicle No. 1, a distance of 120 ft. ahead of No. 2. All dimensions between vehicles represent the distances between the fronts of vehicles. Vehicles Nos. 4 and 5 are oncoming vehicles in the opposing lane

of traffic, No. 4 being the vehicle met by No. 3 before encroaching on the left lane, and No. 5 being the first oncoming vehicle met by No. 3 after completing the maneuver. The space between No. 4 and No. 5 represents the "hole" available in the opposing lane of traffic.

At the top of Figure 3 is shown the speed of each of the five vehicles over the portion of the study section traversed up to this point. Vehicle No. 3 entered the section traveling about 40 miles per hour at a speed of 31 miles per hour and No. 5 at a speed of 47 miles per hour, but No. 5 cannot be seen by vehicle No. 3. The fact that vehicle No. 3 starts to accelerate at this point indicates that the driver has already decided to attempt to pass even though No. 4 is still 166 feet away.

In the second position, occurring 1.7 sec. later and shown by Figure 4, the fronts of No. 3 and No. 4 are parallel; No. 3 has accelerated to 36 miles per hour and is now 74 ft. behind vehicle No. 2. It is

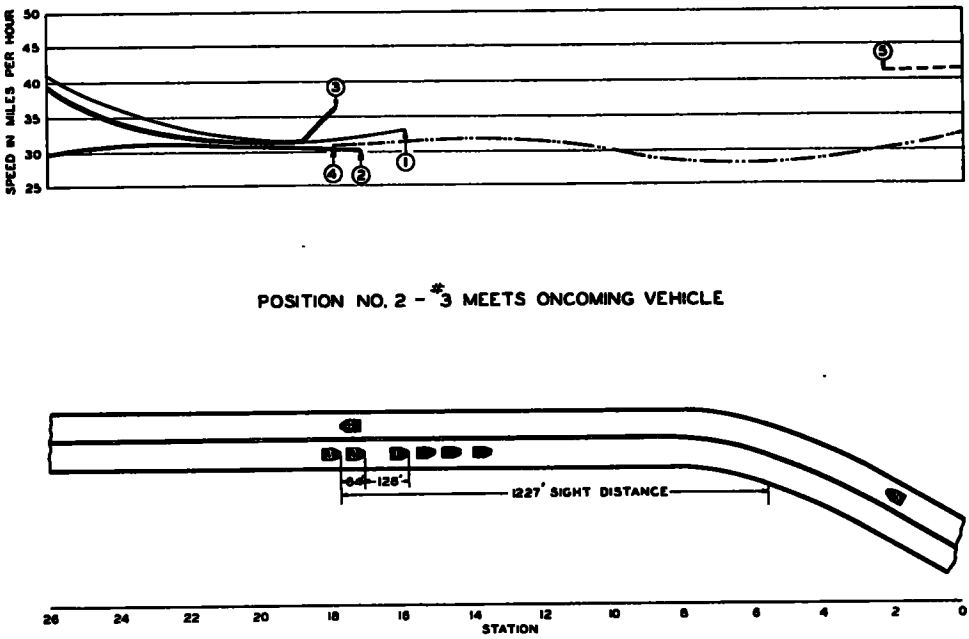


Figure 4

but has been required to slow down to 31 miles per hour, the approximate speed that No. 2 has maintained. (Table 3 indicated that 53.7 percent of the observed passing maneuvers were started after the passing vehicle had slowed down to the same speed as the vehicle to be passed.)

The data sheets for another passing maneuver indicate that vehicle No. 1 has just finished passing No. 2 and has also slowed down to a speed of about 31 miles per hour. Vehicle No. 4 is approaching immediately after this instant that the driver of vehicle No. 3 has his first opportunity to enter the left lane without hindrance from oncoming traffic (vehicle No. 5 still being out of sight).

In the third position, occurring 1.1 sec. later and shown by Figure 5, vehicle No. 3 first encroaches on the left lane while 63 ft. behind vehicle No. 2 and traveling 8 miles per hour faster than the vehicle to be passed.

In the fourth position, taking place 2.7

sec. later (Fig. 6), No. 3 is entirely in the left lane for the first time and is still 37 ft. behind vehicle No. 2. In the meantime vehicle No. 2, which has been traveling at a uniform speed throughout the first 900 ft. of the section, starts to accelerate. The driver evidently doesn't like the idea of being passed or unintentionally steps on the accelerator. He can't accelerate very long without hitting No. 1 but he can reduce the space between his car and vehicle No. 1 so that the driver of the passing vehicle has decided not to try to pass more than one vehicle and is now cutting back into the "hole" between No. 1 and No. 2. This occurs 2.7 sec. after No. 2 and No. 3 were parallel. Vehicle No. 3 is traveling 10 miles per hour faster than vehicle No. 1.

Figure 9, the seventh position, 2.6 sec. later, shows that the passing vehicle completed the maneuver when 107 ft. from the oncoming car. After returning to the right lane, the speed of No. 3 decreased

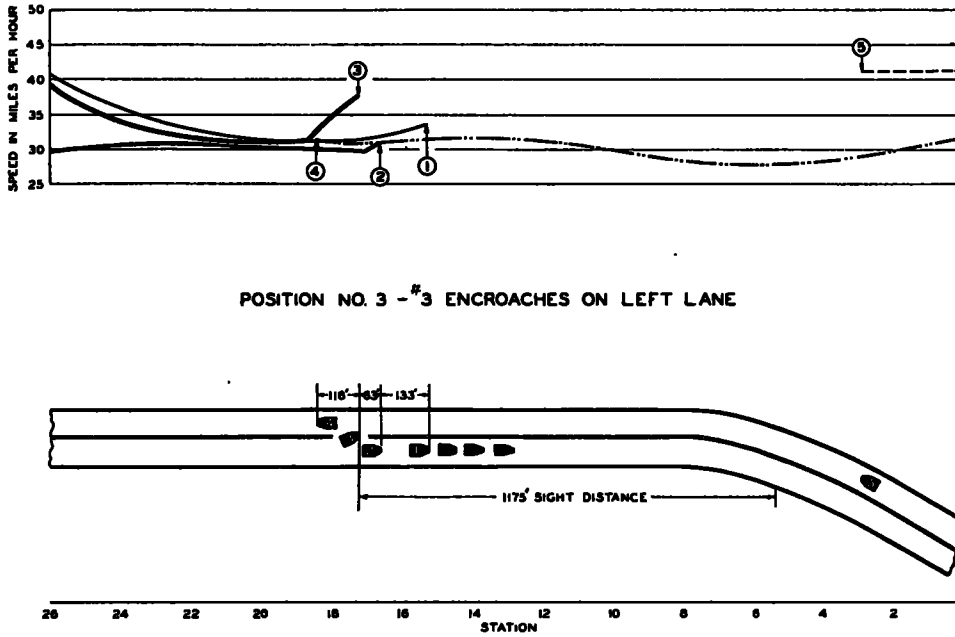


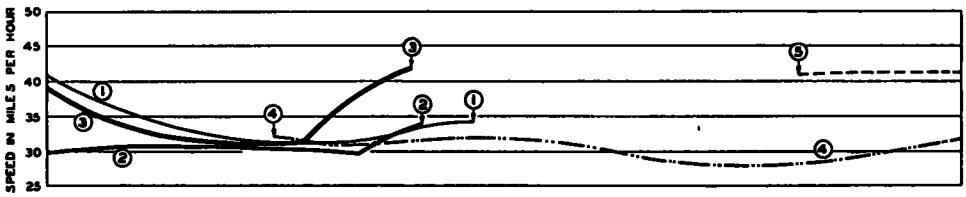
Figure 5

No. 3 will be required to crowd his way back to the right lane. Neither No. 2 nor No. 3 can as yet see the oncoming car No. 5.

Position No. 5 (Fig. 7) occurs 2.9 sec. after position No. 4, and the passing vehicle is now even with No. 2 and is no longer accelerating. Vehicle No. 2 has decelerated a little and the oncoming vehicle No. 5 is now in view.

From Figure 8, representing the sixth position of the maneuver, it is seen that until it was below the speed of vehicle No. 1 and then increased to the same speed. All vehicles slowed down slightly going around the curve.

The acceleration and deceleration curve for the passing vehicle during the maneuver, as shown in Figure 9, indicates a maximum acceleration of 2.3 miles per hour per second. This is considerably lower than any rate that would be assumed in calculating the minimum passing distance under similar conditions.



POSITION NO. 4 - #3 ENTIRELY IN LEFT LANE

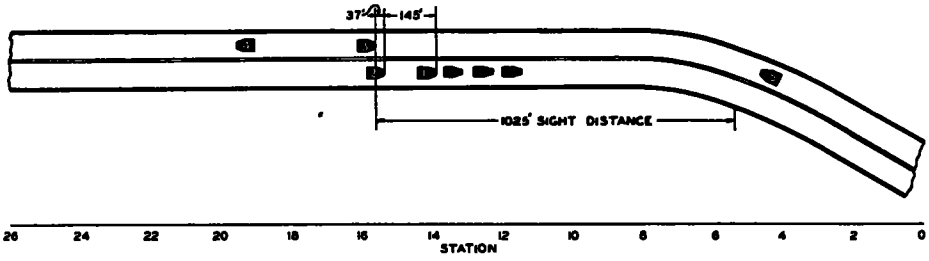
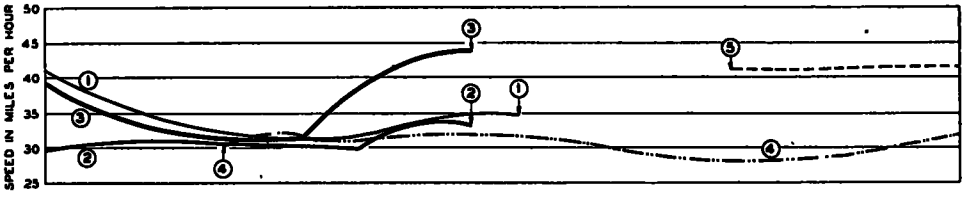


Figure 6



POSITION NO 5 - #3 EVEN WITH PASSED VEHICLE

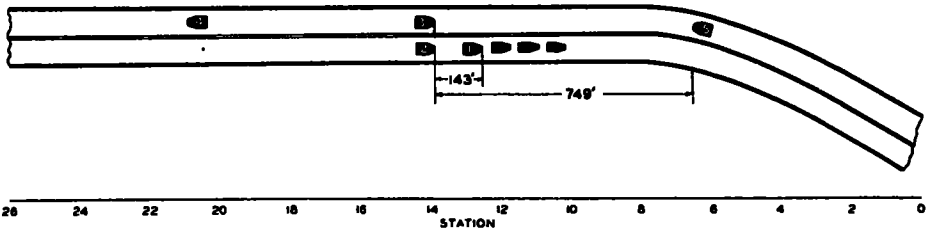
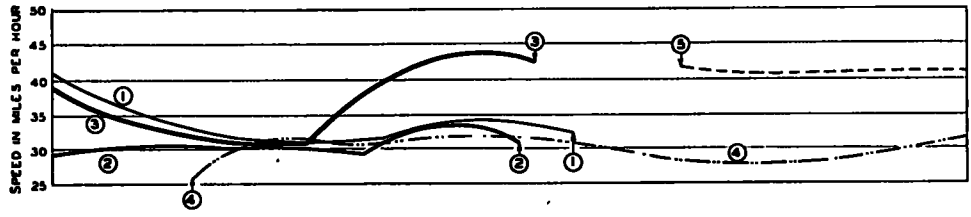


Figure 7



POSITION NO. 6 - #3 STARTS TO RETURN TO RIGHT LANE

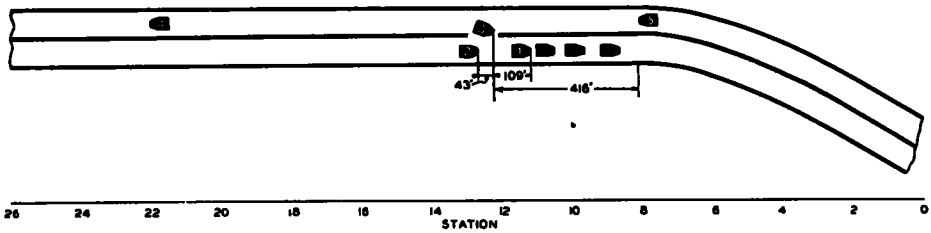
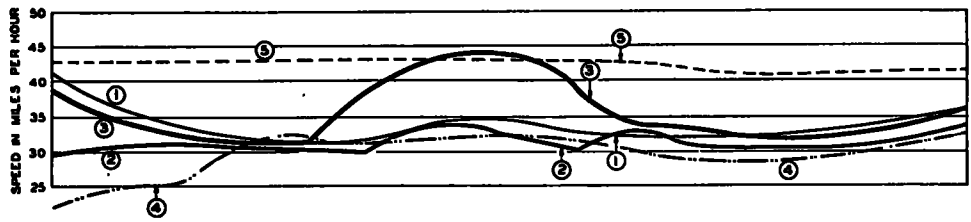


Figure 8



POSITION NO. 7 - #3 BACK IN RIGHT LANE

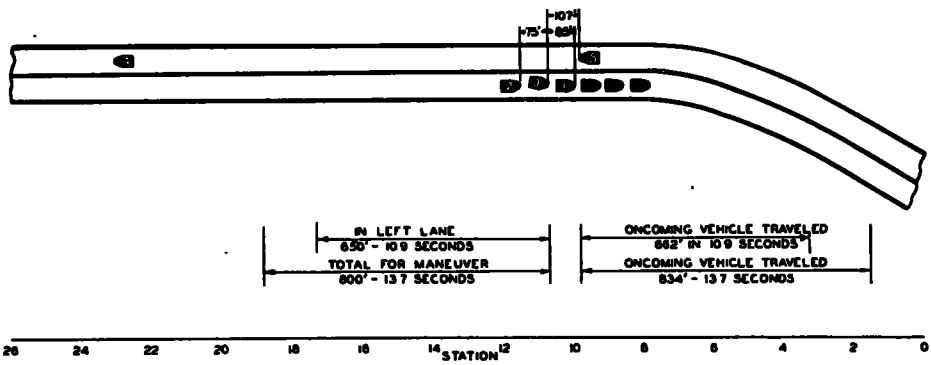


Figure 9

It required 13.7 sec. or 800 ft. to complete the maneuver. The passing vehicle spent 10.9 sec. and traveled 650 ft. in the left lane. The approaching vehicle traveled 834 ft. during the maneuver and 662 ft. while the passing vehicle was in the left lane. The net result of the passing maneuver is that vehicles No. 2 and No. 3 have reversed their respective positions. Providing there are sufficient passing sections and "holes" in oncoming traffic along the remaining portion of this highway, vehicle No. 3 might be able to pass two or three more of the cars of this group before getting to the next town, thereby arriving a little sooner than if no attempt were made to pass. It may be possible for No. 3 to pass all the preceding vehicles in this group at the next section of highway with a long sight distance, providing oncoming traffic does not interfere. Vehicle No. 3 could then continue at the desired speed until catching up with the next group of cars and might reach its destination somewhat sooner than by staying in line. It is evident that this section of highway with a sight distance of 1,900 ft. provided practically no relief from restricted travel conditions for this particular vehicle.

Such a detailed analysis of each passing necessarily takes more time than it would to obtain only such factors as the average vehicle speeds and the total passing distances, but the relative value of the results will more than compensate for the increased work.

There are so many variable factors involved in each passing that it would require almost an unlimited number of passings to obtain a representative sample of each type of passing maneuver possible by different combinations of variables. As the same variable appears in many different types of passing maneuvers, a breakdown of each passing into its component parts will permit a representative sample of each variable to be obtained from a much smaller total number of passings than if the breakdown is not made. The variables may then be recombined to form composite passings covering all types. It is in this respect that the method of analysis, made possible by the complete field records, will differ from other analyses of passing distances and practices that have been made.

The determination of distances involved in a passing maneuver is a relatively simple, though laborious, operation, but the determination of the effect of highway alinement and of driver psychology upon future design requirements is far more difficult. Even though sections of sufficient sight distance to complete individual passing maneuvers in safety are provided, they will not serve their purpose unless the drivers take advantage of their opportunities to pass. Figures 10 and 11 have been constructed from the data for single passings that took place at one particular study location, where vehicles traveling in either direction had maximum and minimum sight distances of 1,900 and 200 ft., respectively. To eliminate a number of variables, only the passings in which the passing vehicle started the maneuver while traveling at the same speed as the vehicle to be passed have been used.

In Figure 10, the passings have been classified by the maximum distance that the driver of the passing vehicle could see that the oncoming traffic lane was clear at the time he encroached on it. Three of the drivers encroached on the left lane when this distance was between 1,800 and 1,900 ft.; 8 encroached when this distance was between 1,700 and 1,800 ft., etc. In 85.5 percent of the passings, the distance was over 1,000 ft. and no maneuvers were started when this distance was less than 500 ft.

Sixteen vehicles encroached on the left lane before reaching the point of maximum sight distance. Fourteen were within 100 ft. of this point and 2 were between

100 and 200 ft. away. As the measured sight distance decreased to 250 ft. immediately before reaching the point of maximum sight distance, the eyes of the drivers of the 16 vehicles were either more than 4½ ft. above the pavement, or the drivers started the passing maneuvers knowing or hoping that the sight distance would increase while they still had a chance to return to the right lane in case an oncoming vehicle came into view. All 16 vehicles reached the point of maximum sight distance before getting entirely in the left lane.

The passings in each of these distance groups have been divided by a heavy horizontal line into two groups. The sight distance limited the distance that

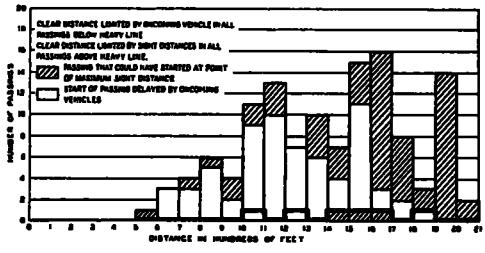


Figure 10. Maximum Distance That Drivers of Passing Vehicles Could See That Left Lane Was Free of Oncoming Traffic at the Time They Entered the Left Lane During the Passing Maneuver (Including Only Those Vehicles That Slowed Down to Same Speed as Vehicles to be Passed Prior to Starting the Passing Maneuver on Study Section Number 1).

the driver could see a clear left lane in all the passings above the heavy horizontal line. There was an oncoming vehicle in sight when the driver encroached on the left lane in all the passings shown below the heavy line. In only six of the passings did the driver encroach on the left lane when there was an oncoming vehicle in view and not a passing was started when the driver could see an oncoming vehicle within 1,000 ft. With no oncoming vehicle in view, passings were started when the sight distance was as low as 500 ft. but relatively few were started below 1,000 ft., there being a marked drop at this point.

The passings are further classified into two groups as determined by whether or not the driver had to wait for an oncoming vehicle before starting the maneuver. All of the passings that are cross-hatched represent maneuvers in which the passing vehicle was immediately behind the vehicle to be passed upon reaching the point of maximum sight distance and could have started the passing maneuver immediately. The areas that are not cross-hatched represent passings that could not have been started at the point of maximum sight distance, because the driver had to wait for oncoming traffic to pass before encroaching on the left lane. Sixty-one (49.2 percent) of the 124 passings that were made, could have been started immediately upon reaching the point of maximum sight distance. In the other 50.8 percent, the passing vehicle waited for an oncoming car after reaching the point of maximum sight distance before starting to encroach.

This one figure in itself contains an enormous quantity of information regarding driver behavior. The number of drivers that did not attempt to pass until after reaching a point where the sight distance was considerably less than it was where they could have started to pass is surprisingly large and offers excellent data for a study of combined perception and judgment time. One driver waited until the clear distance ahead decreased from 1,900 ft. to less than 600 ft. and then started the passing maneuver. He was fortunate, and completed the passing before an oncoming car came into view.

The horizontal scale of Figure 11 shows the clear distance ahead that the driver of each passing vehicle had at the time the maneuver was completed. For all the passings shown below the heavy

horizontal line, this distance represents the clearance between the vehicle that had completed the passing and the first approaching vehicle in the opposing lane of traffic. For the passings above the heavy line, no oncoming vehicles were in view, so the horizontal scale represents the sight distances when the maneuvers were completed. In this figure, as in Figure 10, the passings that are cross-hatched represent maneuvers that could have started at the point of maximum sight distance. In those that are not cross-hatched, the driver had to wait for an oncoming vehicle before encroaching on the left lane.

Figure 10 showed that an oncoming vehicle was in view at the start of the

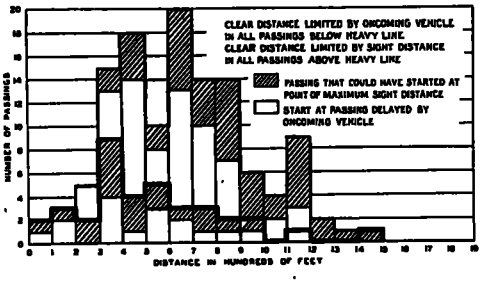


Figure 11. Maximum Distance That Drivers of Passing Vehicle Could See That Left Lane Was Free of Oncoming Traffic at Time Passing Maneuver was Completed. (Including Only Those Vehicles That Slowed Down to Same Speed as Vehicles to be Passed Prior to Starting the Passing Maneuver on Study Section Number 1).

passing in less than 5 percent of the maneuvers. Before the return was made to the right lane, there was an oncoming vehicle in view in about 30 percent of the maneuvers. There is a marked drop in the number of maneuvers that were completed with either a sight distance of less than 300 ft. or a clearance from the oncoming vehicle of less than 300 ft. Two of the five passings that were completed with an oncoming vehicle less than 200 ft. away could have been started when the maximum sight distance was available. The clearances for the vehicles involved in the two maneuvers below 100 ft. were 34 and 49 ft. At the time these vehicles started to return to their right-hand lane, the distances to the oncoming vehicles were 876 and 228 ft. respectively. The first vehicle could have started the maneuver when the sight distance was 1,900 ft. but did not enter the left lane until the sight distance had decreased to 1,675 ft. The second vehicle had to wait for an oncoming vehicle to pass and started the maneuver with a sight distance of 625 ft.

By similar analyses of the passings that occur on different sections, it will be possible to determine the relative effectiveness of different alinements in providing for passing requirements.

Up to the present time, the analyses dealt primarily with the passings that actually occurred on the study sections. Of equal importance are the passings that the drivers wanted to make but did not attempt because they felt that the available sight distances or the "holes" in the opposing traffic were not of sufficient length to complete the passing maneuvers in safety. No attempt has been made to take this particular information from the records, but it is believed that it can be obtained. So far, it has been possible to obtain all the factors that have seemed important in a study of passing distances.

From the rather meager results that have been presented it can be seen that these studies provide, for the first time, accurate information on what actually takes place in a stream of moving traffic. They are certain to provide extremely valuable information regarding the causes of accidents even though none may actually occur during the studies.

METHODS OF MEASURING JUDGMENT AND PERCEPTION TIME IN PASSING ON THE HIGHWAY

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SYNOPSIS

In determining the minimum sight distance which allows safe passing, it is necessary to know the interval needed by the driver to decide whether it is safe to attempt a pass. This interval must be added to the actual passing distance or time used in calculating the minimum tangent or straight stretch necessitated. Design engineers have recognized this judgment and perception time interval and various estimates as to its magnitude have been made.

Two methods of measuring this interval are described. The first, which employs movies from directly above the test location, is capable of greater accuracy. The second method, which consists of timing by an observer with a stop watch, is rougher but easy and economical. The observer judges the physical layout to determine the beginning of the sight distance, and the driver registers his decision by stepping on the accelerator. A series of results are given for this rough check method.

The results showed two judgment and perception time figures. Both were selected to include 80 per cent of the cases. The first was 3.5 seconds and the second was 2.8 seconds. The former is more applicable to a tangent between curves and the latter may apply to other problems such as those relating to intersections.

A factor which is of interest in connection with the problem of minimum safe passing sight distance is that of judgment and perception time of the average motorist when passing another car. In a previous study of overtaking and passing distances,[1] the actual distances required were related to speeds of operation. In that study it was pointed out that for the determination of the length of tangent required to allow safe passing a clearance time should be allowed between the oncoming vehicles at the moment of completion of the pass. Also, at the start of a following or accelerative type of pass, a time allowance is necessary for the driver to estimate speeds and make his decision as to whether or not he will attempt to pass.

This "Judgment and Perception Time" has been recognized by leading design engineers and certain assumptions have been made as to its magnitude.[2] To the best of our knowledge, however, actual measurements of the time required have not previously been made.

The present paper, therefore, reports two methods which may be used for the determination of the judgment and perception time used by the ordinary driver as he rounds a curve and enters a tangent on which he has sufficient distance to pass. The first of the two methods allows greater accuracy but requires more expense and time and labor. Preliminary trials of this method have been carried out to a sufficient extent to specify the procedure which will be practical. The second method is less accurate but is

[1] Matson, T. M. and Forbes, T. W. "Overtaking and Passing Requirements as Determined from a Moving Vehicle." *Proceedings*, Highway Research Board, Vol. 18, p. 100, 1938.

[2] Barnett, J. "Needed Research for the Determination of Sight Distance." *Proceedings*, Highway Research Board, Vol. 17, p. 111, 1937.

* With the assistance of the 1940 class of Fellows of the Bureau for Street Traffic Research.

We are indebted to officials of the Connecticut Highway Department and to Major Johnson and Captain Generous of the 43rd Aviation Division, Connecticut National Guard for their cooperation in preliminary testing of the air method.

quick and practical and probably gives a fair approximation of the length of time in question. A series of observations from one section of the country obtained by this method will be presented.

THE PROBLEM

In hilly country where curves are fairly close together, the minimum length of tangent necessary to allow safe passing may be thought of as bounded by a curve on both ends. As shown in Figure 1A, if a driver is following a slower vehicle around the curve and entering the tangent, there will be a short distance, *ab*, during which he will not be able to start his pass although he has reached the point from which sight distance would be measured and found adequate for passing. This distance, *ab*, represents the time consumed by this second driver in pulling his vehicle slightly to the left in order to see around the vehicle ahead, plus the actual time required to perceive and judge the relationship of on-coming cars and speeds in order to decide whether or not to pass. For practical purposes, therefore, this time value represents the additional distance which must be added to the actual distance required for the pass when considering minimum passing sight distances. For our purposes, therefore, it is possible to lump the judgment and perception time and the time for lateral translation.

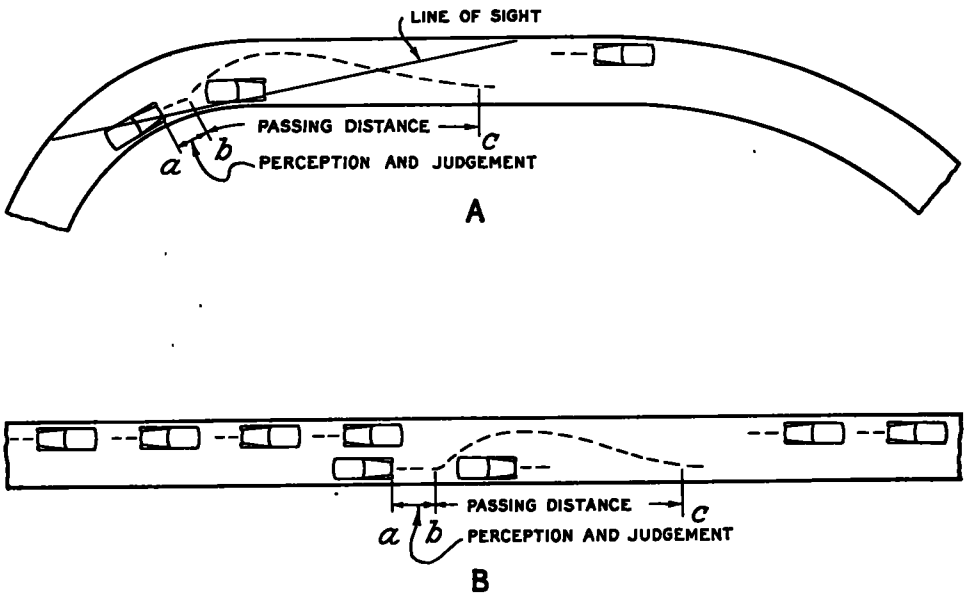


Figure 1. Measurement of Judgment and Perception Time

A—As applied to minimum sight distance between curves
B—As applied to heavy traffic conditions on tangents
ab = Lateral travel plus judgment and perception time
bc = Passing distance

A somewhat similar situation as far as the driver's reaction is concerned sometimes occurs in the middle of a long tangent, as shown in Figure 1B. The following driver has been forced back into line by a long string of on-coming vehi-

cles. At the moment illustrated in the figure he has reached a point abreast of the last car before a gap in the on-coming line. In order to see ahead, he thereupon pulls slightly to the left and makes a judgment somewhat similar to that in Figure 1A, after which he initiates his pass.

The time corresponding to the distance "ab" in Figure 1B therefore corresponds to a similar judgment and perception time, but the lateral translation phase is very much reduced. Times so determined, therefore, would be expected to be more nearly the true perception and judgment time alone and to be shorter than those determined in the curve or crest situation of Figure 1A. They are not so directly applicable to the problem in hand as those of 1A, but may be of value for other purposes.

For a determination in situation B, it is necessary that the driver shall have been forced completely back into line behind the vehicle he is following. If he has been riding the center line in such a position that he can see ahead at all times, he will already have made his judgment and decision before the on-coming car pulls abreast and there will therefore be no zero point for the time measurement.

AIR PHOTO METHOD

A method has been devised and tried out by which determinations of judgment and perception time may be made from the air by means of a moving picture technique. A test location is chosen where drivers are bottled up and trying to pass a car ahead. It must be of such alignment that passing sight distance becomes available to the driver suddenly at a definite point. In general the method consists of taking serial photographs from directly above the test location. The plane speeds and camera characteristics must be carefully chosen. Time values can then be obtained in terms of the number of movie frames registering the movement in question and distance values may be obtained by measuring the projected picture one frame at a time.

Distance Values

In order to obtain distance values from photographs, a given dimension which appears in the photograph must be known. If the edge of the highway is sharply delineated, the width of the highway furnishes such a dimension. Delineation is best obtained by a white edge such as concrete against a dark shoulder. With the aid of such a known dimension, any slight variations in the size of the picture due to changes in altitude of the plane may be easily corrected by adjusting the size of the projected picture. Such adjustment, of course, is easily attained by moving the projector nearer or farther from the screen.

Another necessary feature is some sort of a definite landmark along the highway which appears in each frame of the movie. By referring all measurements to such a landmark, the longitudinal distances involved can be determined. A screen ruled off in squares representing distance values is of convenience for measurement.

Speed and Altitude Relationships

We will now consider relationships between speed and altitude as they bear on the type of aircraft used and related problems. Figure 2A illustrates the relationship of the minimum safe flying speed to the altitude which must be used, if the camera is to be kept fairly vertical above the vehicles being observed. Obviously a slower type of aircraft can fly lower in inverse proportion to speed with the same angular variation from the vertical.

Figure 2B illustrates the relationship of plane speed to car speed and to the parallactic error involved. From the figure it will be seen that the error e is a

function of d_3 and of the altitude h. If we assume a car traveling at 30 miles per hour, 10 sec. of travel represents approximately 450 ft. If 1 sec. of this or 45 ft. of distance is taken as the quantity under measurement 1 percent error would be 0.5 ft. From previous studies of overtaking and passing it is thought that the time represented by ab will fall within the first 5 or 6 sec. On this basis 1 sec. would be a conservative value on which to base such a longitudinal error.

Assuming a height of 6 ft. for the vehicle and an error e of 0.5 ft. the longitudinal distance, it can be shown that

$$d_3 = 0.08h \qquad (1)$$

Also

$$d_3 = \frac{1.47t(V_2 - V_3)}{2} \qquad (2)$$

where t is the time of travel v_2 is the velocity of the plane and v_3 the velocity of the car. Substituting

$$h = 9.25tV_{diff} \qquad (3)$$

From equation 3 the values of Table 1 have been obtained using 10 and 20 mile differential speeds. Thus, if a car is traveling 40 miles per hour and the aircraft at 60 miles per hour, in order to

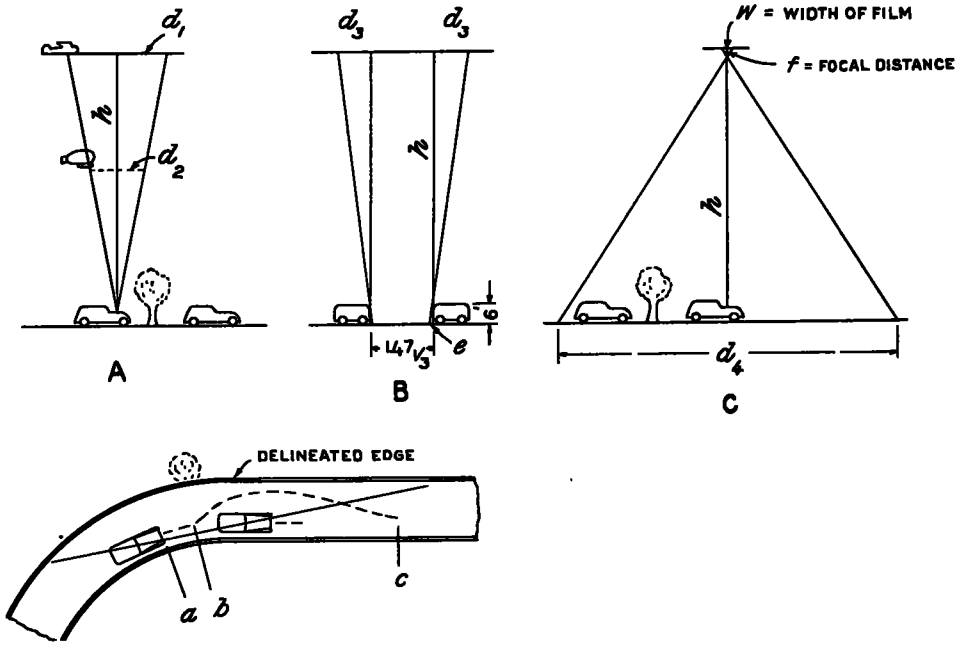


Figure 2. Measurement of Judgment and Perception Time Air Photo Method

a, b, c as in Figure 1
A—Speed relationships
B—Parallactic error
C—Camera field

TABLE 1

ALTITUDE FOR ONE FOOT LONGITUDINAL ERROR

Speed differential m.p.h.	Altitude for 1.0 ft. total error (e=0.5 ft. Fig. 2) and duration of travel of	
	1 second	10 seconds
10	92.5 ft.	925 ft.
20	185.0 ft.	1850 ft.

record the path of the vehicle within a total error of 1.0 ft. for 1 sec. an altitude of 185 ft. would be required and for 10 sec. 1850 ft. as a minimum altitude. For slower speeds of the plane the error would obviously be smaller at a given altitude. For a 10 mile differential it will be seen that anything over 925 ft. would be satisfactory. Since average speeds on most eastern highways are generally between 40 and 45 miles per hour with a range of ±20 miles per hour it is recommended that an aircraft speed of about 40 or 45 miles per hour be used. Such a speed is safely and conveniently obtainable only with a blimp.

Since the lateral speed can be made practically zero the error in the lateral measurements will not depend to any extent upon the relative speeds of the plane and car. It will, however, depend upon the magnitude of the photographic image obtained and on the accuracy with which a projected picture can be measured. This means that the record should be made from as low an altitude as is compatible with safety and which will still give sufficient photographic field to include the amount of travel during the interval under observation.

Camera Characteristics

Figure 2C considers the camera field in relation to altitude. It will be seen that

$$d_4 = \frac{w}{f} h \qquad (4)$$

where d_4 is the travel to be observed, w is the width of the film, f is the focal distance of the camera and h is altitude. Assuming the use of ordinary 16 mm. movie camera with a 1-in. lens equation (4) becomes approximately

$$d_4 = 0.64h$$

Assuming maximum of 10 sec. travel at 60 miles an hour d_4 becomes 900 ft. and the altitude 1400 ft.

It thus becomes evident that a 16 mm. camera with a 2-in. telephoto lens can be satisfactorily employed at 1000 ft. altitude as far as camera field is concerned. To eliminate fuzziness from vibration, a camera with variable shutter aperture is necessary in order to obtain a rapid exposure without the use of high speed photography and the attendant rapid use of film. Thus a clear picture can be obtained at 8 frames per second even with some vibration. A trial roll taken with an ordinary 1-in. lens from a plane moving at 90 miles per hour at 2000 ft. indicated the need for these camera characteristics. A speed of 8 frames per second gives an accuracy of time measurement approximating the minimum possible driver reaction and therefore should be sufficient. A camera with a special constant speed governor is essential.

Measurement

As previously noted, for easy measurement the film may be projected on a screen which has been marked off in distance values. Previous experience indicates that where the projection appears slightly blurry, a fairly accurate measurement may be made by spotting the picture from a moderate distance.

Devices are obtainable which allow the marking of an individual frame on the film. Such a device might be used for obtaining the time values in terms of the number of frames utilized.

Advantages and Disadvantages of the Method

The advantages of the method include easy motility, the ability to observe traffic without disturbing it, the possibility of observing the general run of drivers on the highway, and the possibility of accurately determining the point at which the sight distance at the location begins. This point may be spotted

on the ground or obtained from highway department maps for given test locations.

The disadvantages consist of the expense of aircraft and film, the time and manpower necessary for measuring of records, and the necessity of a subjective judgment from the projected picture of the point at which the driver starts his pass. (Point b in Figure 1.)

Other Uses and Recommendations

It is felt that this method will be of use not only for the problem in hand but also for the measurement of the operation of vehicles by the public in relation to many other problems. For instance, the measurement of convenient values of deceleration, comfortable rates of lateral translation of vehicles and many similar problems might be attacked by the same method.

Equipment has not been available to date for the satisfactory gathering of data on the problem of judgment and perception time, except to show the feasibility of certain altitudes and the need of slow velocity aircraft. From the above considerations and the trial pictures obtained, the best combination appears to be pictures at 8 frames per second with a 2-in. telephoto lens (16 mm. movie camera) from a blimp operating at 40 to 45 miles per hour, at 1000 ft. altitude.

STOP WATCH METHOD

A simple method has been used which is probably less accurate but quick and convenient. It was noticed by accident on an inspection trip, that a driver who is following another vehicle and is intent on passing it steps down with marked definiteness as soon as he makes his decision. In other words, he steps hard on the accelerator at the point "b" of Figure 1. It is therefore possible for an observer riding with him to pick out this point from the sound of the motor and time it with a stop watch. The method thus utilizes the reverse reaction to that employed by others in measuring visibility under headlights.[8]

The observer must however determine the point "a" at which the sight distance would normally begin by means of his own judgment of the alignment and physical layout. This judgment by the observer is obviously a source of error, especially if it depends upon the estimates of one person alone. A number of observers were therefore employed. The error involved will be considered later.

Procedure

For the first group of observations, the drivers were told only that a study was being made of passing and that they were to "drive as if they were in a moderate hurry." They were therefore unaware of the details of the experiment. The observer timed the interval from a to b (Figure 1A) and also timed the complete pass with a split second stop watch. In this fashion no indication was given to the driver that interest centered on one portion of the pass alone.

Before the second group of observations the group of drivers and observers had heard a discussion of the problem and they were therefore aware of its exact nature. However, they had not yet seen the results obtained from the first observations nor discussed the magnitude of the judgment and perception time value.

A third group of observations was obtained from six women drivers "without knowledge" of the exact problem. For the first two groups of observations 19 observers and 20 different men drivers took part. The male group were for the

[8] Roper, Val J. and Howard, E. A. "Seeing With Motor Car Headlamps." *Transactions*, Illuminating Engineering Society, Vol. 33, p. 417, 1938.

most part traffic engineering fellows studying at Yale University.

Times were recorded by tabulation on a convenient card form in 0.5 sec. intervals. Columns were arranged for right curve and crest, left curve, and tangent conditions.

The observations were obtained on a two-lane highway (some concrete and some black top) between New Haven and Hartford, Connecticut, during off-peak hours of the afternoon (2 to 5 P. M.). This highway alignment includes frequent horizontal curves, and the speed of traffic probably corresponds to the Connecticut State average fairly well (40 to 45 miles per hour).[4]

Results

The results of observations made by 19 observers on the driving of 20 men and 6 women have been lumped together in order to obtain the effect of an average judgment of the location of point "a" of Figure 1A.

In treating the results a choice must be made between considering each driver as a unit to obtain an average for each person and treating each individual observation as a unit in a large total distribution. Table 2 shows the characteristic record of one driver to illustrate the fact that the record of each driver may show as wide a range as the total time range of the whole distribution. This means that the effect of different road and judgment conditions is of more importance than the individual characteristics of each driver. For this reason the observations obtained from all drivers will be combined as a single distribution of individual time determinations.

Figure 3 shows a series of such distributions lumping all observers and all drivers, but analyzing the effect of various experimental conditions. For example, Figure 3A shows observations on men drivers without knowledge of the experiment. The left hand figure represents curves and crests and the right hand one, tangents.

[4] Tilden, C. J., *et al.* "Motor Vehicle Speeds on Connecticut Highways." Committee on Transportation, Yale Univ., 1936.

TABLE 2

SAMPLE RECORD OF ONE DRIVER—STOP-WATCH METHOD

Time *sec.*	Observed times		Tangents
	Right curves and crests	Left curves	
0—.4	...	...	...
.5	4	1	1
1.0	5	4	1
1.5	2	1	2
2.0	3	1	...
2.5	...	...	...
3.0	2	...	...
3.5	...	...	...
4.0	1	2	...
4.5	...	...	...
5.0	...	...	...
5.5	...	...	...
6.0	...	...	...

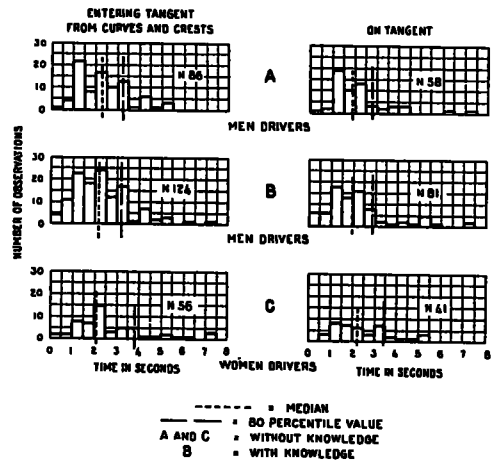


Figure 3. Effect of Knowledge of Details of Experiment, Judgment and Perception Time, Stop-Watch Method.

Effect of Knowledge of Details of the Experiment

Before it is legitimate to throw together the results of different sets of observations, it is necessary to examine the effect of other variables. As noted above, the term "without knowledge" is applied to observations obtained when the driver knew only that a study of passing was being made. After acting as driver, he was then given an explanation and acted as observer for other members of the group. The caption "with knowledge" indicates observations taken after discussion of the problem in detail. Such discussion might conceivably influence subsequent driving behavior. However, this was not the case.

Figure 3A shows the results obtained "without knowledge" and Figure 3B "with knowledge." There is apparently less skewing of the distribution in the second case, but there is very little difference in the median and 80 percentile values. We may therefore conclude that there was no appreciable effect.

Women Drivers

Figure 3C presents the results obtained with the six women drivers. The median value is very nearly the same as that for Figure 3B, but the 80 percentile value is somewhat higher. The number of drivers is so small that too much reliance should not be placed on this series of results, but they do suggest that women drivers may require a slightly longer judgment and perception time in this highway situation to accommodate 80 percent of the group.

Direction of Curve

It was thought that possibly left curves might present a different situation than right curves and crests, since in a left curve the sight distance opens up more quickly. That is, the width of the highway, being to the left of the driver, tends to give him a continuously broader sweep of vision.

Therefore in Figure 4 these two conditions have been compared using the records of men drivers only. There is no appreciable difference in the median or 80 percentile values of these two conditions, for those time observations which were obtained. However, observers reported that it was difficult to obtain observations on left curves because in many cases the driver had made his decision by the time the sight point was reached. The fact that only 33 observations were obtained for left curves as against 91 for right curves and crests probably reflects and corroborates this report.

We may conclude, therefore, that in many cases the left curve vision opens up so gradually that the driver's decision is already made at the starting point "a," but that under conditions where oncoming vehicles or other complications necessitate a real judgment by the driver at point "a" the time involved is of the same general range and magnitude as in the right curve and crest situation.

Curves and Crests Compared with Observations on Tangents

From the considerations above it now appears valid to lump all observations for men drivers on curves and crests and similarly to lump observations on long tangents corresponding to situation B of Figure 1. Such distributions of total observations for men drivers are shown in Figure 5.

Both distributions are again definitely skewed and the 80 percentile value for the curve and crest situation is about 0.4 sec. greater than that for observations on tangents. Table 3 shows that the actual 80 percentile judgment and perception time are 3.25 sec. and 2.85 sec. respectively.

These times corroborate the expectation that more lateral transition of the vehicle was involved in the curve and crest situation than in the tangent observations. Thus the figure of 2.85 sec. probably represents an 80 percentile value for a more nearly pure judgment and perception time while 3.25 sec. is more applicable for the practical design of tangents between two curves.

In examining the mode of each distribution it should be noted that the figure shows a definite tendency by observers to record times on even seconds rather than in the half-second category. This tendency is a well known one in psychological circles wherever estimates or reports of a number of persons are

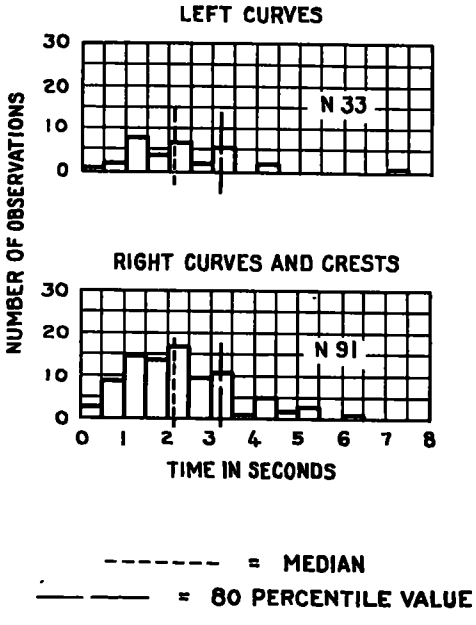


Figure 4. Left Curves Compared with Right Curves and Crests, Judgment and Perception Time, Stop-Watch Method.

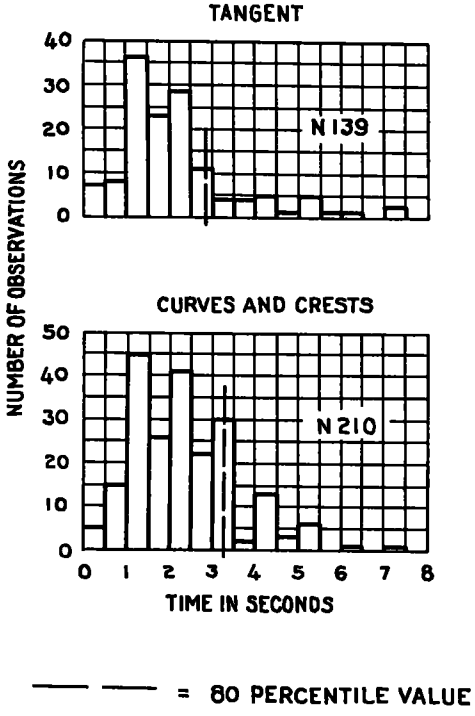


Figure 5. Tangent Compared with Curves and Crests, Judgment and Perception Time, Stop-Watch Method.

TABLE 3

SUMMARY OF JUDGMENT AND PERCEPTION TIME—STOP-WATCH METHOD

	Median		80 percentile	
	Curves and crests	Tangents	Curves and crests	Tangents
Men drivers	*Sec.*	*Secc*	*Sec.*	*Sec.*
"Without knowledge of experiment"	2.25	1.85	3 25	2.75
"With knowledge"	2.10	1.90	3.20	2.85
Total Distribution	2.20	1.90	3.25	2.85
Women drivers	2.05	2.20	3.80	3.45

involved. It is probable that the discrepancy within the 1-sec. intervals should be somewhat equalized and that for this reason the mode should fall at approximately 1.5 sec.

It is of interest that the mode should be similar in the two cases, and probably indicates that this is a minimum judgment and perception time value in this situation. Values below 1 sec. are relatively infrequent and fall in the range which is known to represent brake reaction times. Therefore these short times probably represent judgments which were partially made before the observed zero point on the highway was reached.

Sources of Error in the Stop-Watch Method

Two main sources of error are inherent in the stop-watch method. The first of these is that the observer's own perception time is involved. However, this is not so serious an error. The observer must judge the location of point "a" in terms of the physical layout and alignment. However, he makes this estimate in a relatively gradual manner since he can observe the unfolding outline of the curve or crest, and start his watch at the first point which would give a clear, safe passing sight distance. Thus all that is involved is his simple perception and reaction time at this point. It is known that ordinary stop-watch reaction times run about 0.1 sec. and this error should therefore be negligible in our result.

A more serious error rests on the fact that the judgment of the actual point "a" may involve a very substantial distance error. Such errors might be made in either direction and if a sufficiently large number of observers and observations is involved, a normal distribution of these errors should be obtained. If so, this error should balance out.

Adequacy of the Sample

Although the number of *observations* should be sufficient to give a stable determination, it must be borne in mind that our results represent a relatively small number of *drivers*. They also represent a sample from one type of terrain and fairly heavy traffic conditions.

Our observers were probably pretty well qualified to make the determinations in the second group of observations, Figure 3. In the first group they may not have been quite so well qualified as observers, but they were better qualified as a representation of the general public since their knowledge was less complete. In any case this factor does not seem to have been of great importance.

The drivers in our sample were probably better than average on the whole since most of them were professionally interested in traffic.

For these reasons larger samples of observations on the general driving population should be obtained by the air photo method described in the first part of this paper. Furthermore, a sample should be obtained for different conditions of traffic density and terrain in different parts of the country if a more accurate and exhaustive determination is desired.

Our sample should, however, be adequate to give an approximate idea of the time values involved.

Comparison with Reaction Times

As noted above, the modal judgment and perception time is seen from Figure 5 to fall somewhere near 1.5 sec. This gives us a value which may perhaps be compared with certain average times which have been obtained in other studies. Thus an average complex or choice reaction time with steering and brake apparatus has been determined at approximately .60 to .75 sec. and a simple reaction time at about .5 sec.[5] A modal judgment and perception time of about

[5] DeSilva, H. R. and Forbes, T. W. "Driver Testing Results," Bureau for Street Traffic Research and W. P. A. of Mass., Boston, 1938.

1.5 sec. would seem to be a reasonable continuation of this series of average times.

However, it should be pointed out again that the 80 percentile value rather than the modal value should be used where possible for design and control calculations.

Practical Application of Time Values

It has been pointed out elsewhere that the use of the 80 percentile value takes care of the large proportion of the general run of drivers.

Therefore, for the determination of minimum passing sight distance and minimum length of tangent between curves, the value of 3.25 sec. obtained from men drivers would seem to be applicable. The fact that the judgment and perception time from the women drivers was approximately half a second longer suggests that 3.5 sec. would not be too great a value to adopt where practically feasible from other considerations. For a 50 mile per hour design speed, such an interval adds 258 ft. to the actual passing distance. The 80 percentile passing distance for this speed was shown to be about 1200 ft. in our previous study.[1] An addition of over 20 percent is necessary at this speed, therefore.

Judgment and perception time may be viewed as a constant value, and passing times were shown to be proportional to speed. It follows that the proportion added by the judgment and perception time will be greater at lower design speeds. The shorter judgment and perception time of 2.8 which represents the 80 percentile point for the tangent observations may be of interest in other problems connected with design and control devices.[6] It is probably closely related to the time for complex judgment which is necessary at intersections and in similar traffic situations where no extra interval for lateral translation is involved.

Both the intervals of 3.5 sec. and 2.8 sec. are somewhat greater than some estimates which have been made for these values.

[6] Barnett, J. "Needed Research for the Determination of Sight Distances at Intersections." *Proceedings*, Highway Research Board, Vol. 18, p. 76, 1938.

SUMMARY

1. Two methods were described for measuring judgment and perception time in passing. The first, an air photo method, allows greater accuracy, while the second is a rough check stop-watch method.

2. From a consideration of relative speeds, altitude and camera characteristics as well as results of exposing a trial roll at 2000 ft. altitude, the use of 16 mm. movies at 8 frames per second, with a 2-in. telephoto lens at 1000 ft. from a blimp operating at 40 to 45 miles per hour was suggested.

3. The rougher method consisted of timing the judgment and perception time with a stop-watch. The observer riding in the car judged the point at which sufficient sight distance became available from his own observation of the physical layout. The time at which the driver had made his decision to pass was almost invariably indicated by "gunning" of the motor.

4. Nineteen different observers were employed and a total of over 200 observations to minimize errors in determining the sight distance zero point. This error would be expected to balance out.

5. Results from the rough check method indicated that when drivers follow a vehicle around a curve and attempt to pass on entering the adjoining tangent, a judgment and perception time of 3.25 to 3.5 sec. should be allowed for them to judge speeds and make their decisions. This figure included 80 percent of the observations. Twenty men and six

women drivers were tested. The latter tended toward somewhat longer time values.

6. Part of this figure undoubtedly represents lateral travel of the vehicle. A judgment and perception time (80 percentile) value of 2.85 sec. resulted where less lateral travel was probably involved. This figure is probably more nearly a measure of judgment and perception time alone and may be of value for other traffic problems.

7. For more accurate determinations, observations should be made on a larger number of drivers under different traffic densities in different parts of the country by the air photo method.

8. The 3.5 sec. time when added to the actual passing distance at 50 miles per hour represents the addition of approximately 20 percent in distance. This interval is of importance in the computation of minimum safe passing sight distance, a factor controlling to a large extent the desirable minimum length of tangent between curves on highways. The importance of the interval has been recognized by leading design engineers, but measurements of it have not been previously obtained.

DISCUSSION ON METHODS OF MEASURING JUDGMENT AND PERCEPTION TIME

DR. BRUCE D. GREENSHIELDS, *College of the City of New York:* The methods of measuring judgment and perception time in passing on the highway, given by Dr. Forbes, bring to mind the method used in Michigan in 1933 to study the minimum spacing between vehicles. In this study a movie camera arranged to take pictures at intervals of about 1 sec. was used. The exact interval found most convenient was 1/88 of a minute, for the distance in feet traveled during this time interval equals the speed in miles per hour. The method is fully described in the PROCEEDINGS OF THE HIGHWAY RESEARCH BOARD, Vol. 13, pages 362-396 (1933).

During the past summer, in Ohio, in connection with a study of the "Marking of No-Passing Zones," some time-motion pictures were taken. For ease in analysis, such pictures should be numbered. This was accomplished by including in each picture a view of a Veeder-Root Counter which was mounted about 20 in. in front of the camera and brought into focus by means of an auxiliary lens. This counter was actuated by the same electric-release that operated the camera. The electric release timer and the counter were operated continuously, while the camera was switched on only while vehicles were passing. This conserved film. Figure 1 shows the bulletin board used to identify each film and two consecutive pictures. The camera was an Eastman Cine Kodak Special, 16 mm. The electric release is made by the Eastman Company.

The point at which the driver starts to accelerate, as explained by Dr. Forbes, gives the point at which his perception-reaction is completed. This point, at favorable locations, can be found by means of the camera, although it is not mounted directly above the roadway. If the field of view is extensive enough, a view from the side gives the speed or change of speed.

An unobstructed view from the inside of a curve, such as is shown in Figure 1-A of Dr. Forbes' paper, would be especially favorable for pictures taken from the ground or a slight elevation, such as might be obtained from a truck tower. Located on the inside of the curve, the distance from the camera to any point on the roadway would be about the same. The displacement of the cars against the

background gives the speed so that the movement of the camera does not affect the accuracy of the observations if the shutter opening is made brief enough to avoid blur. This means that the camera could be rotated on its tripod to keep the car or cars in view.

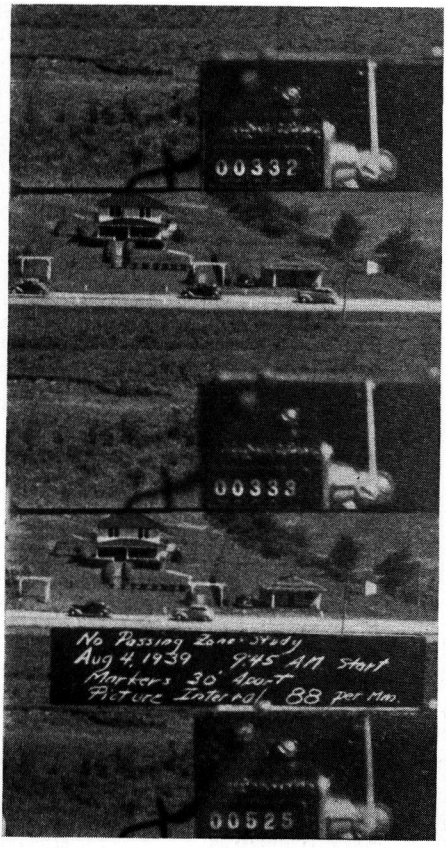


Fig. 1

The position of the vehicle on the opposite lane at the instant of passing can be obtained from its speed when it appears in the pictures. This idea is explained in "Distance and Time Required to Overtake and Pass Cars," PROCEEDINGS OF THE HIGHWAY RESEARCH BOARD, Vol. 15, pages 332-342 (1935).

It is pointed out in the 1933 paper that the spacing between vehicles depends upon reaction time, or more correctly, on perception-reaction time. The average time was found to be about $\frac{3}{4}$ of a second. A re-study of the data shown on page 391 of the 1933 Proceedings, gives the 80 percentile value as 2 sec. at 5 miles an hour, and 1.3 sec. at 39 miles an hour. These values agree closely with the median values given by Dr. Forbes.

DOCTOR FORBES: The discussion by Dr. Greenshields raises an interesting point as to the relationship of this time interval to that previously found for the spacing of vehicles on the highway. Dean Johnson, Thompson and Hebden, Greenshields, and others, deserve credit for the use of moving picture analysis of various phases of traffic behavior, and Dr. Greenshields especially for introducing reaction time factors.

Although the method reported by Dr. Greenshields in his 1933 article was considered by us for the judgment and perception time problem, it presented certain difficulties. With a camera operated from a position on the ground at one side of the highway and trained at right angles to the highway, it is very difficult to record certain very important items. For instance, a driver may be so placed that he can look down the center line of the highway and in this case there can be no determinable zero point for his judgment time. Again, the placement of vehicles ahead of him may be such that his view is completely blocked for a long period of time and therefore he cannot be expected to make any judgment as to whether or not he can pass until such a situation has cleared up. A third difficulty is that some drivers accelerate rather slowly, but do turn their vehicle laterally in starting to pass. If the motor is gunned even momentarily, it can be heard by an observer riding in the car, but it would be rather difficult to detect a slow acceleration accurately from the recorded motion of the car. In place of

such a determination of acceleration, the lateral path of the vehicle can be obtained from our air photo method and we feel that this would be the more reliable.

The most important point, however, regards the conditions of observation in the 1933 study made by Dr. Greenshields of the spacing of vehicles. If our understanding of this report is correct, these times were obtained for the spacing of vehicles as they were operating on open highway. This means that the drivers may or may not have been trying to pass the car ahead, and the supposition is that the majority of them were merely following in line.

These spacings were shown in his report to be explainable in terms of the length of the vehicle plus a 5-ft. spacing between vehicles (found at zero velocity), and an added spacing when moving at moderately high speeds which corresponded to a brake reaction time of 0.75 sec. (p. 392-394). This interpretation of the spacings which were found seems valid and corroborates the assumption that no judgment of the type we are discussing in our paper was included in the time interval. (Complex reaction and judgment times are known to be longer than ordinary brake reaction time.) It therefore does not seem allowable to re-interpret the time values obtained from the spacings of Dr. Greenshield's study as judgment and perception times.

If the judgment and perception time necessary for the driver to make up his mind about a passing situation were included in Dr. Greenshields' experimental conditions, his spacings in terms of time should have been considerably greater. In our determinations on tangents, for instance, we sometimes obtained 2½ or 3-sec. times for the driver to make up his mind and yet, during this period, he might be following the car ahead at a headway as short as 1.0 sec. This is another indication that the spacing of ordinary traffic represents a different condition than does the situation where a driver is trying to pass and requires a certain amount of time to make up his mind whether or not it will be safe. The two time values would, therefore, not be expected to be comparable.

Incidentally, it seems a little hard to justify the comparison of an 80 percentile figure from the 1933 study with a median (50 percentile) figure from our determinations. It will be noted that our 80 percentile values were 2.8 and 3.25 sec. respectively—rather larger than Dr. Greenshields' 1.3 sec. value.

In summary, it seems that the conditions under which Dr. Greenshields' 1933 time values were obtained do not satisfy those for the determination of judgment and perception time and that the particular form of moving picture recording which he used would not be as suitable for our purposes as the methods suggested. However, the question of the relation of judgment and perception time to stopping brake reaction time and the spacing of vehicles is of interest and we are grateful to him for raising the question. The two psychological functions involved are distinctly different.

FINAL REPORT
JOINT COMMITTEE ON ROADSIDE DEVELOPMENT
AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS AND HIGHWAY RESEARCH BOARD

H. J. NEALE, *Chairman*

Landscape Engineer, Virginia Department of Highways

SYNOPSIS

The Committee announces the dissolution of Joint Committee on the accomplishment of its task of integrating the work of the various agencies, and of defining the various factors in a complete roadside development program. The work properly divided and correlated is to be continued by appropriate committees of the Association and the Board.

Brief résumés of the reports of the subcommittees on Highway Types and Roadside Areas, Slope Erosion Control, Plant Ecology, Zoning, Roadside Economics and Education and Public Relations are given.

In 1932 the American Association of State Highway Officials and the Highway Research Board jointly appointed a Committee on Roadside Development. At that time, it will be recalled, the first serious consideration was being given to the development of the nation's roadsides, with only a few States organized for handling the work. To many it was an æsthetic gesture, devoid of economic background. The interest manifested by the sponsors of the committee and by the Bureau of Public Roads in this new field of endeavor inspired a nation-wide interest in the improvement and development of the roadsides that has now permeated every State. The Committee was constituted primarily to study every factor involved in a complete roadside development program.

It has functioned under six project-committees, involving major research of Highway Types and Roadside Areas, Erosion, Plant Ecology, Zoning, Roadside Economics, and Education and Public Relations. Members of these subcommittees have labored diligently during the seven years of their existence, and the reports published by the Highway Research Board during the last three years readily emphasize the valuable contributions their studies have furnished the highway engineers.

At a meeting of the Joint Committee held in Minnesota in August of this year, a resolution was unanimously adopted recommending the discharge of the Joint Committee, based on the fact that the primary objectives for which the Committee was appointed had been accomplished, up to a point where the purely technical research problems and the administrative factors could be more efficiently handled by separate committees. The Committee on Roadside Development of the American Association of State Highway Officials has agreed to undertake a study of the general phases which do not involve intensive research. To this end, its Chairman has appointed sub-committees in every district of the Public Roads Administration. These subcommittees will function directly with the district engineers and the staff of the Public Roads Administration and the landscape engineers of the States involved. They will study the various problems pertinent to the geographical area of the district, based on nation-wide mutual problems. Coincidental with the work of this committee, it is recommended that the Highway Research

Board appoint a committee[1] to study and report on the purely research problems. In this way, the Committee of the American Association of State Highway Officials will refer research problems to the research committee, and by the same token the research committee will refer administrative problems to their committee. It is felt that by this procedure a more complete and intensive study of the economic, utilitarian, and æsthetic problems involved in the development of the nation's roadsides will be more completely covered. Therefore, this report will constitute the last official report of the Joint Committee.

It is interesting to note at this time a gradual integration of the original recommendations of this committee into the roadside programs of the various States. In brief, these involve—more adequate right-of-way width, together with absolute control being vested in the various highway departments; conservation of natural roadside vegetation; competent landscape personnel in every State; the use of adequate ground cover and mulch to prevent erosion; the featuring of appropriate plant materials at intersections, for avenue effects, for snow breaks, for architectural features; the promotion of wayside areas; the development of roadside springs; the opening and development of safety turnouts and vista sites; the control of outdoor advertising; the encouraging of cleaner, more sanitary, and landscaped roadside stands; and in general the promotion of any activity that will improve and give the public better use of the roadsides. The roadsides are but the frame of a continuous panorama landscape and as such their development must be devoid of artificial effects and replete with natural settings. The gradual advancement of these factors has been accomplished through a better appreciation of their values by the public, and by the constantly increasing cooperation of the highway engineers.

Your Committee, from time to time at its various meetings and through the medium of its reports, has stressed all of these features, substantiating their claim that developed roadsides make for economy and safety, and that natural roadsides make for beauty.

The various trade journals, publications, and the press of the nation have been most cooperative in publishing many valuable contributions concerning Roadside Development activities. No doubt, the most outstanding contribution of the past year was by the *Contractors and Engineers Monthly* in offering a trophy as a national award for the contractor or his superintendent that made the greatest contribution toward Roadside Development in the United States during 1939. Supplementing this award, they have offered four sectional awards. This has awakened the interest of the contractor, to the higher ideals incident to a more finished appearance and protection of the roadsides of what has previously been a typical "no man's land." Not only have the contractors who have been nominated for these awards fulfilled their obligations to the State in adhering to the specifications and completing the same to the best of their ability, but they have provided ancillary note that will redound to their credit in future years.

A brief résumé of the Project Committee's reports is given herewith. The full reports of the Committee[2] will be published and will be available for distribution.

[1] A committee of the Design Department of the Highway Research Board on *Roadside Development* has been appointed with the following initial membership: H. J. Neale, chairman, F. A. Aust, P. H. Elwood, Wilbur Simonson, F. E. Clements, G. B. Gordon, J. L. Wright, Arnold Davis, A. R. Nichols, Frank Brant, Hugh Pomeroy.

[2] Report of Joint Committee on Roadside Development for 1939, Bulletin, Highway Research Board.

HIGHWAY TYPES AND ROADSIDE AREAS

The third annual report of the Sub-committee on Highway Types and Roadside Areas contains a brief review of the two previous reports of 1937 and 1938 as a background for the present report. The objectives of the Sub-committee are outlined in a general way as the basis for a possible program for further detailed analysis on a regional administrative basis.

The 1937 report emphasized the need for a broader approach to the highway cross-section problem so that the right-of-way as a whole is considered a part of its surroundings.

The 1938 report further emphasized this relationship of the highway to its surroundings. The foundation of a well-balanced design is an adequate width of right-of-way, with roadbed, roadside, and adjacent lands all united in proper relation. The composite diagram in the report showing the "Trend in Expanding Widths of Divided Highways in Relation to Rights-of-Way" presented basic information recognizing the need for those elements that contribute to a balanced program of highway development.

The present report includes typical highway cross-sections now in regular use in one specific administrative region or highway district to show the gains which have been made in recognition of the many advantages of orderly roadside development through integration of landscape practices including flattening and rounding of slopes in initial highway construction. The cross-sections for the region selected typify the evolutionary progress in the improvement of the actual right-of-way and the relation of the right-of-way to adjacent lands.

In order to provide for factors of safety and recreation the modern highway should include provision for wayside areas in addition to the fundamentals of highway engineering and landscape design.

It is proposed that this single regional presentation serve as a sample for the guidance of further studies along similar lines in every administrative highway region of the United States.

SLOPE EROSION CONTROL

The prevention of erosion along and adjacent to highways is a widespread problem throughout the nation. The soil treatment thereof to prevent erosion has often proved very costly and in many cases has not always been entirely successful. For effective erosion control, the solution of these problems involves engineering factors pertaining to cross-section, methods of drainage, construction of stabilizing structures, and problems of location, and the necessary drainage structures with respect to topography, as well as forestry, agronomy and soil problems dealing with the establishment of conditions most effective for plant growth.

The Project Committee on Slope Erosion has studied these problems as they relate themselves to the three major fields:

(1) Watershed areas adjacent to the highways, both above and below the highway; (2) highway ditches, and (3) cut and fill slopes.

To study these related problems in the different problem areas of soil, climate and topography, 110 cooperative projects have been planned and constructed in 37 States. The work has gone forward according to the outline prepared by the committee. Results of the work, as based on three years' experience, are found in (1) the earlier reports of this committee, (2) the handbook now ready for publication, "Graphic Solution of Ditch Design and Erosion Control," and (3) in the preliminary and final reports of the various projects. The final reports will be digested by the future committee. The

time interval required for the testing and studying before conclusions may be evaluated is a matter of five or more years rather than the short time which has elapsed since these projects have been undertaken.

PLANT ECOLOGY

The Project Committee on Plant Ecology has made extended studies during past years of the plant materials best adapted for highway planting in the various climatic regions. The experience of the State highway departments up to the present time leads to these conclusions:

1. That in each region, existing vegetation along a highway furnishes the key to proper selection of the trees and ground cover plants to be established on each section of that highway.
2. That well-selected native trees and other native and well-naturalized plants can be readily established, and can be maintained after establishment on highway areas at low maintenance cost.
3. That successful and economical establishment of well-selected plant materials, either by artificial seeding and planting or by natural volunteer growth, is largely dependent upon *good original highway location, a well-flattened and rounded cross-section* and upon *restoration of soil* which will support plant growth.

The report of the Project Committee for 1939 covers an analysis of the problem of selection and use of ground cover plants on highway areas with special regard to the Northeastern and Middle Atlantic regions of the United States.

ZONING

Since the purpose of roadside development is to merge the highway into the landscape surroundings, the studies of the Project Committee on Zoning justify the conclusion that comprehensive control of lands adjacent to the highways is essential to protect mutually the highway investment and the economic and scenic values of the adjacent lands. Zoning is an important implement of planning and as such the planning surveys being conducted by the various States will effect more permanency through its use.

ROADSIDE ECONOMICS

The Project Committee on Roadside Economics was appointed for the purpose of assembling data relating roadside development to safety, maintenance cost, and property values.

Since the accident records are not available for a sufficient period of time, and since there are extreme variations in accounting practices, only case history methods of study could be used on selected highway mileages instead of collecting comprehensive data based on entire State highway systems.

The report furnishes interesting data on the cost of various types and classifications of work on which roadside development funds have been expended, and also comparative data concerning mowing operations on improved and unimproved roadsides.

EDUCATION AND PUBLIC RELATIONS

In order to furnish literature concerning activities of the Joint Committee and roadside development operations in various parts of the country, a Project Committee on education and public relations was appointed. The report of this Committee was accompanied with a first issue of the bulletin entitled "Roadsides—A Record of Progress"—the bulletin which the Committee expects to publish quarterly. This Project Committee also sponsored a traveling photographic exhibit in

cooperation with the Public Roads Administration. The Committee reports and cites various articles on roadside development which have been issued during the last year.

In conclusion, the Chairman is desirous of acknowledging the valuable and conscientious cooperation received from the various members of the Committee and project committees, the Director of the Highway Research Board, the Chairman of the Department of Design of the Highway Research Board, the Public Roads Administration, the Soil Conservation Service, the National Park Service, the Tennessee Valley Authority, the United States Forest Service, the various State highway departments, and the many public-spirited citizens who have given of their time and thought toward an economical and efficient program of roadside development.

REPORT OF SUBCOMMITTEE ON HIGHWAY TYPES AND ROADSIDE AREAS

W. H. SIMONSON, *Chairman*

Senior Landscape Architect, Public Roads Administration

SYNOPSIS

The third annual report of the Subcommittee on Highway Types and Roadside Areas contains a brief review of the two previous reports of 1937 and 1938 as a background for the present report.

The 1937 report emphasized the need for a broader approach to the highway cross-section problem. The right-of-way as a whole should be considered a part of its surroundings.

The 1938 report further emphasized this relationship of the highway to its surroundings. The foundation of a well-balanced design is an adequate width of right-of-way, with roadbed, roadside, and adjacent lands all united in proper relation. The composite diagram in the report showing the "Trend in Expanding Widths of Divided Highways in Relation to Right-of-ways" presented basic information recognizing the need for those elements that contribute to a balanced program of highway development.

The present report includes typical highway cross-sections now in regular use in one specific administrative region consisting of Minnesota, Wisconsin, North and South Dakota to show the gains which have been made in recognition of the many advantages of orderly roadside development through integration of landscape practices in initial highway construction. The variable climatic and topographic features of the region are shown in a series of maps.

It is proposed that this single regional presentation serve as a sample for the guidance of further studies along similar lines in every administrative highway region of the United States.

In its first annual (1937) report, the subcommittee recognized as its first step the need for a survey of current practices in highway design which have a basic relation to the landscape development of highways. This report emphasized the need for a broader approach to the cross-section problem, by considering the right of way as a whole in relation to its surroundings. This recommendation questioned the customary practice of thinking of the highway cross-section as covering only certain limited portions of the width, such as the surfacing, shoulders, guard rail, superelevation and widening, and similar engineering details of construction.

Although available information relating to the landscape development of highways within the United States has not been fully correlated for classification purposes, the general survey of the evolution of the highway cross-section in regular construction, as tabulated in the 1937 Report on "The Design of the Highway Cross Section,"[1] may serve as a basis for greater coordination and uniformity for future design and construction practice.

A knowledge of the general trend toward a wider application of the principles of landscape design to typical highway cross-section grading and drainage problems is of practical value to officials who plan local programs of highway construction and maintenance.

The second annual (1938) report supplemented the previous report on the primary two-lane rural highway by covering, in a similar way, "The Sectional Layout of Multiple-Lane Highways."[2]

[1] *Proceedings,* Highway Research Board, Vol. 17, p. 255.

[2] *Proceedings,* Highway Research Board, Vol. 18, p. 190.

In addition, there was appended in the 1938 report of the subcommittee a digest of replies on the design of roadside areas: "Safety turnouts" and "Waysides."[3] This summary defined the purpose, the types and kinds, the size, the location, and the maintenance of "Wayside" facilities, with observed suggestions for application in future development of "Waysides."

These two reports (1937 and 1938) furnish a composite picture of current tendencies in the sectional layout of tomorrow's highways. Construction trends indicate a progressive widening of roadbed surfaces and shoulders, the flattening of crowns and of slopes of shoulders and gutters, as well as the flattening and rounding of cut and fill slopes and increasing right-of-way widths. The facts point toward a better balanced cross-section design with a growing emphasis placed on the landscape development of highways.

The 1937 and 1938 reports furnished a general survey of highway cross-section development over the United States from 1920 to about the present time.

This third report is aimed to be more specific in its application through geographical limitation to one administrative region or highway district of four States, North Dakota, South Dakota, Minnesota and Wisconsin.

The Summer (August 28, 29, 30, 1939) Meeting of the Joint Committee on Roadside Development held at St. Paul and Gull Lake, Minnesota, naturally suggested District 4 of the Public Roads Administration as a logical administrative area for specific study. It is the subcommittee's desire that this report covering a typical region of highway administration will encourage a similar analysis of cross-section practice to be made in each of the 12 Federal highway regions of the United States.

The variable climatic and topographic features in this administrative region of the two Dakotas, Minnesota and Wisconsin furnish a good basis for purposes of analysis and comparison of typical governing conditions which highway design, construction and maintenance must recognize. Data concerning the prevailing characteristics of the four States in District are given on maps as follows:

Figure 1: Topography (Elevation Reference).
Figure 2: Rainfall (Annual).
Figure 3: Soils (Distribution).
Figure 4: Temperature (35°+).
Figure 5: Temperature (55°+).
Figure 6: Temperature (55°−).
Figure 7: Temperature (35°−).
Figure 8: Frost (Fall).
Figure 9: Frost (Spring).
Figure 10: Vegetation (Natural).

Figure 1 indicates that the topography ranges from about El. 699 ft. along the shores of Lakes Michigan and Superior on the east to El. 7,242 ft. at Harney Peak in the Black Hills, west of Rapid City, South Dakota. For the purpose of cross-section analysis this region includes three general classifications of topography or landscape:

1. Smooth or easy topography (relatively level to gently rolling).
2. Moderate or rolling (heavy rolling, or hilly to light mountainous).
3. Rough or mountainous (heavy mountainous).

The average annual precipitation, as shown on Figure 2, varies from less than 15 in. in certain western portions of the Dakotas, where only short grass will grow, to over 30 in. east of St. Paul, Minnesota, where moisture conditions

[3] January 1939 Report by The Joint Committee on Roadside Development, Highway Research Board and American Association of State Highway Officials. (P. 39—Appendix.)

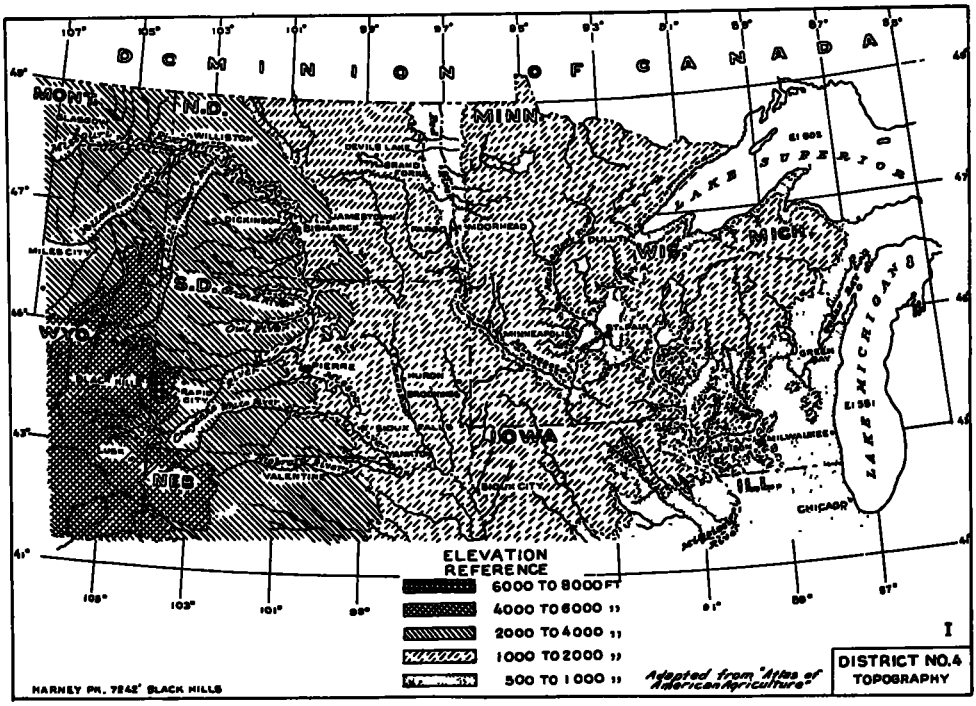


Figure 1

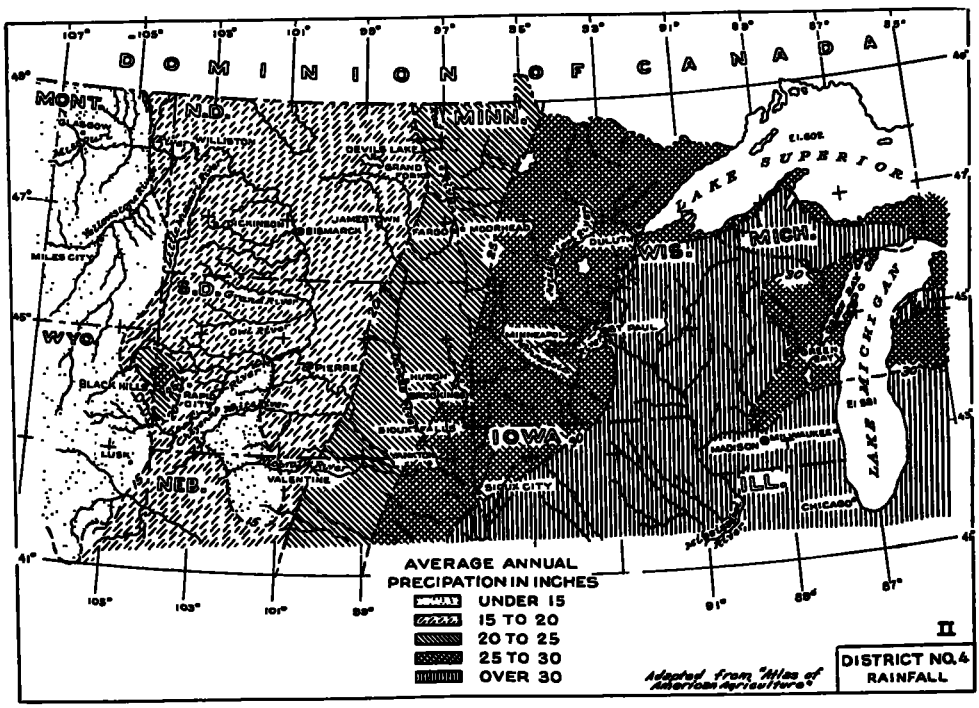


Figure 2

are favorable to deciduous and evergreen tree growth.

Similarly, Figure 3 presents wide distribution of the great soils groups, from the moist and acid re-acting podzol soils of northern Minnesota and Wisconsin, through the brown soils of the northern prairies to the dry sandhills of Nebraska. The typical problems of wind and water

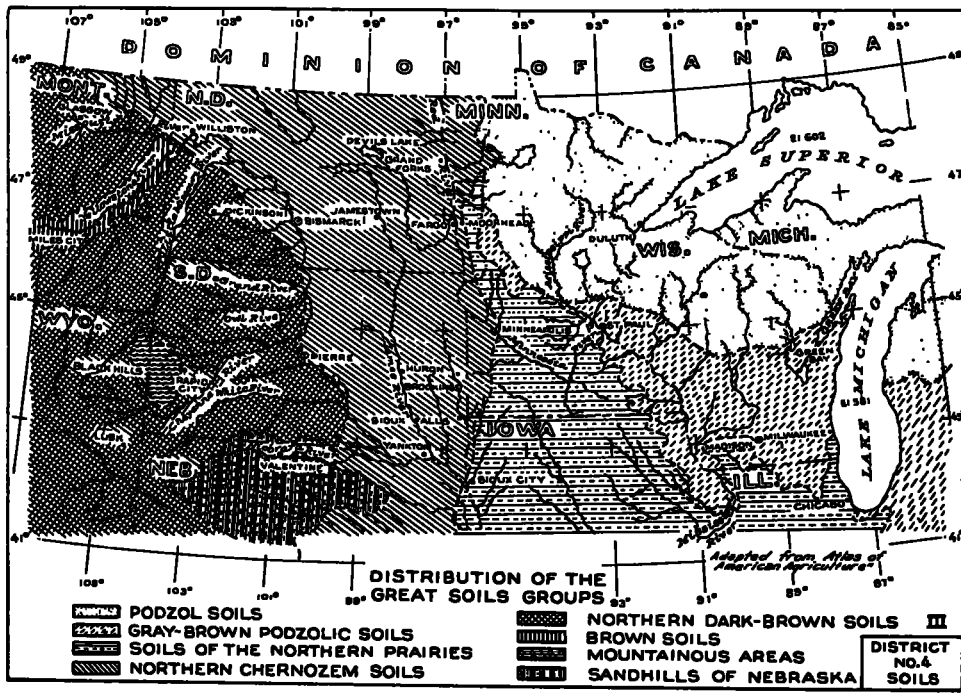


Figure 3

The average date when mean daily temperature rises above 55 degrees, or when plant growth gets active, covers a similar four weeks' period, from about May 1 to June 1 or later, as shown on Figure 5. Highway specifications need to make allowance for such factors.

Conversely, the average date when mean daily temperature falls below 55 degrees, or when plant growth becomes inactive, is shown on Figure 6 to vary from about September 15 to October 15. Thus, the length of the growing season in this region varies from about four to six months, seriously limiting seasonal operations in the colder northern portions to a relatively short working period.

erosion are evidenced by the soils found in these four States.

From north to south in this region, Figure 4 shows that the average date when the mean daily temperature rises above 35 degrees represents a spread of about four weeks, from March 15 to April 15. Freezing weather limits certain highway construction operations, especially the more seasonal items of work, like seeding, sodding and planting, as well as bituminous and cement operations.

On Figure 7, the average date when mean daily temperature falls below 35 degrees is indicated from before November 1 to after November 16. The fall

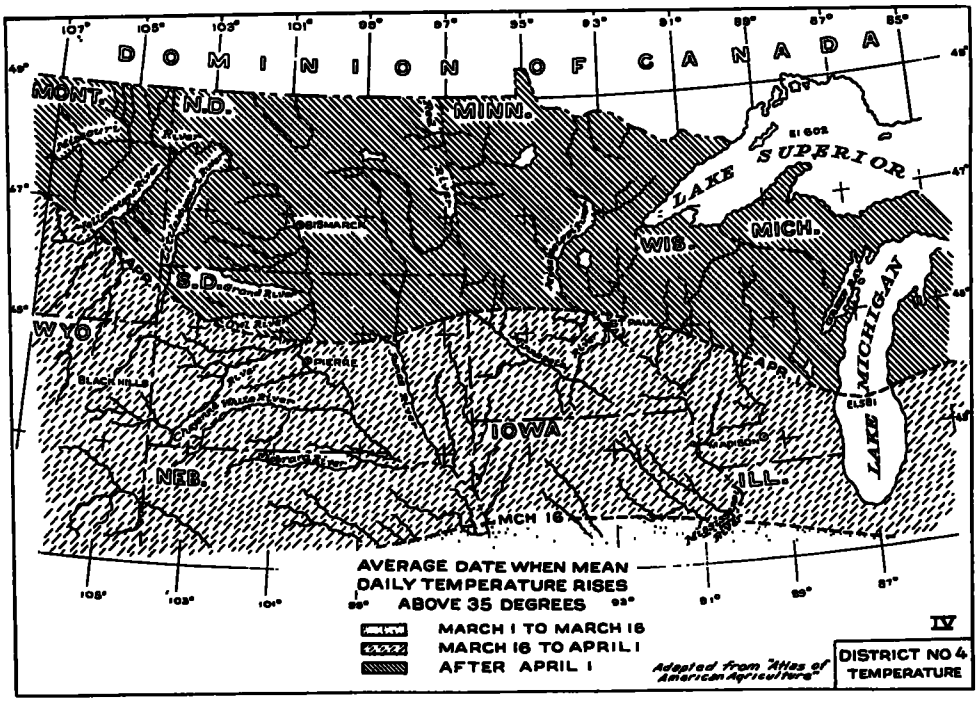


Figure 4

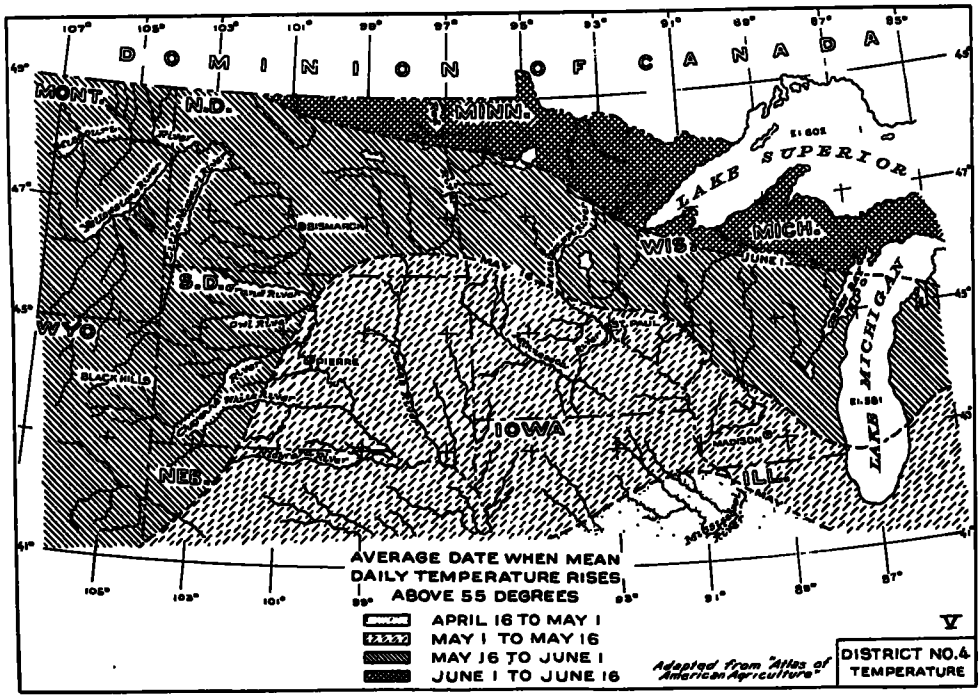


Figure 5

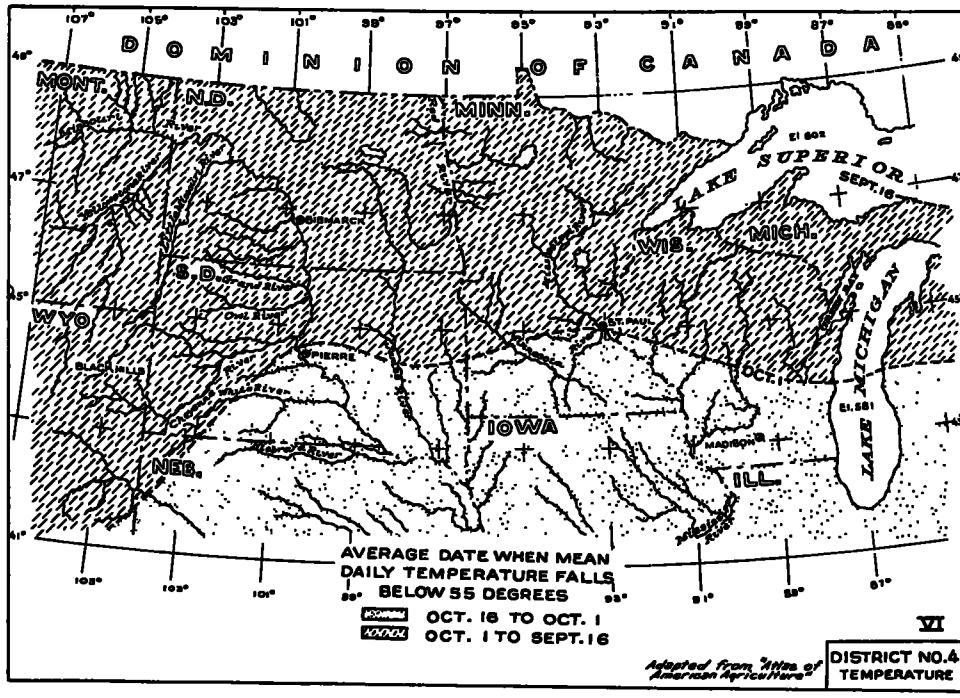


Figure 6

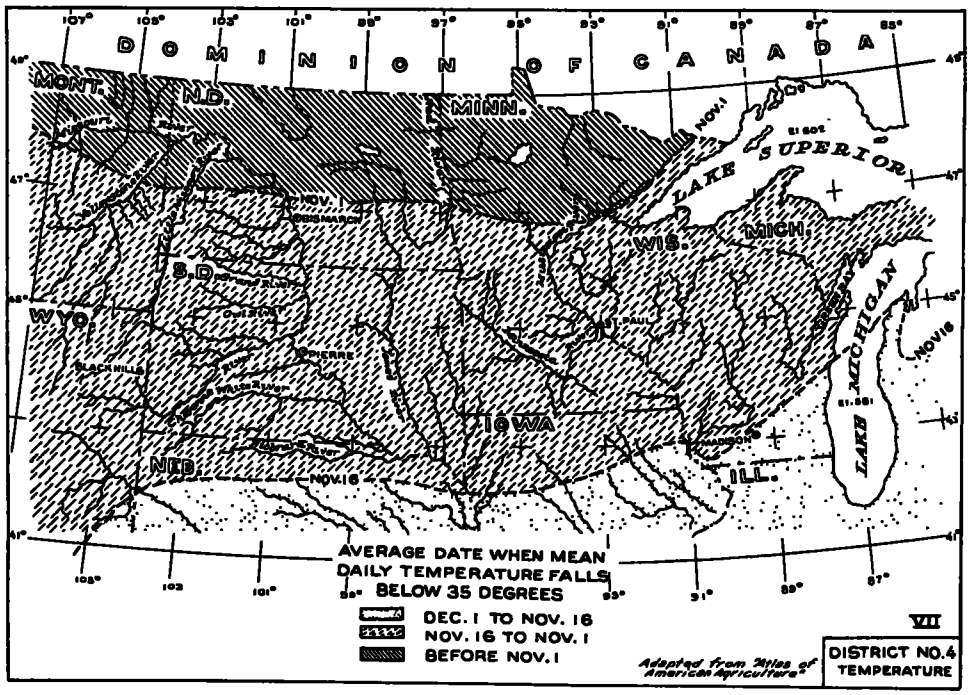


Figure 7

closing date for strictly seasonal items of work appears to be less variable than the opening dates in the spring.

According to Figure 8 the average dates of the first killing frost in the fall range from about September 10 in the north to about October 10 in the south; and according to Figure 9, the average dates of the last killing frost in the spring vary from about May 1 to about June 1 of the prairie grassland in the mid-region to the short grass of the dry plains grassland of the western portion.

It is generally recognized that these varying factors control plant growth in a region and, therefore, influence roadside planting work. It is not so generally recognized, however, that these same factors influence road design as well as landscape design, although nonseasonal high-

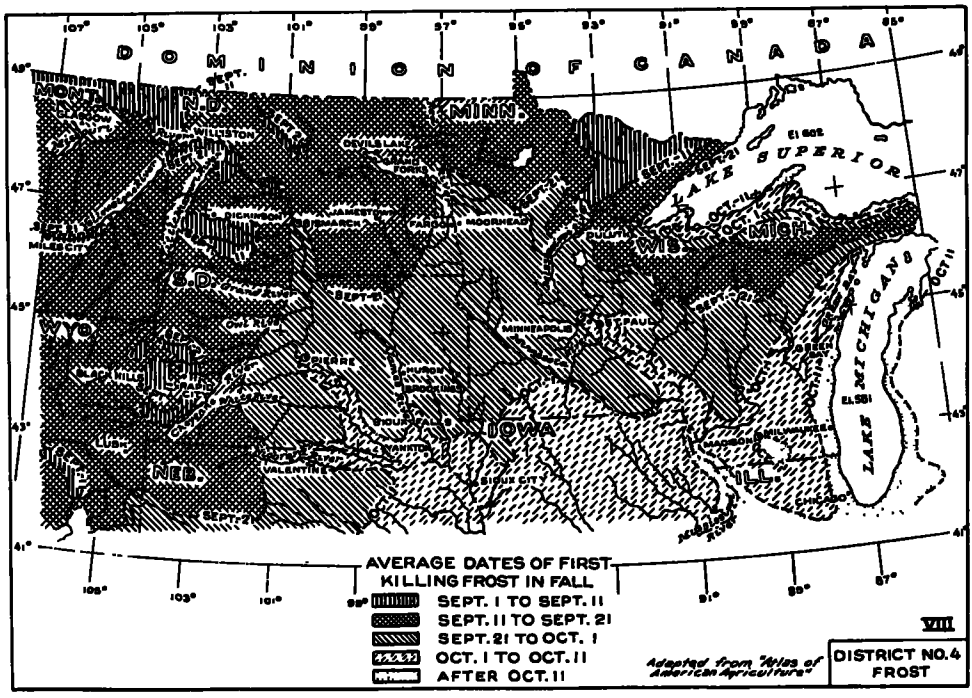


Figure 8

for the region. The comparative analysis of topographic and climatic factors which govern road design and landscape design operations must be the basis for the intelligent application of technical research data.

The product of all these climatic and topographic features is the natural vegetation of a particular locality or region. The moisture loving tree growths of the eastern portion (both deciduous and evergreen) are modified to the tall grass way operations are not so subject to these factor variations as are seasonal roadside activities.

Figure 10 furnishes a clear picture showing the extreme variation in the resulting products of different combinations of soil and climate.

While the map information accompanying this report may serve as a basis for research analysis in a wide range of roadside development studies, the present report is confined to an analysis of

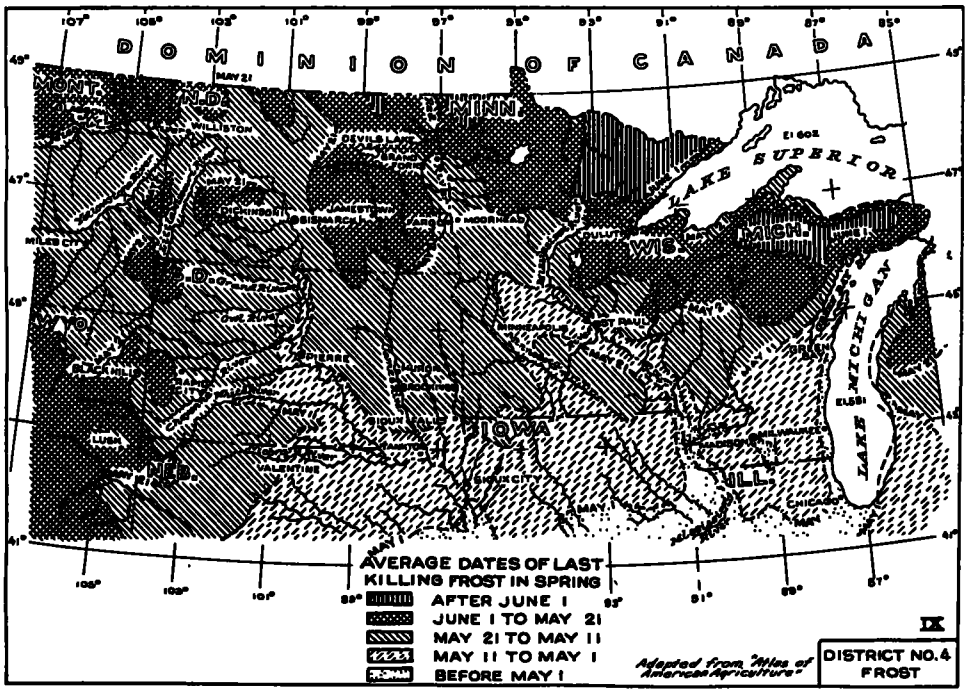


Figure 9

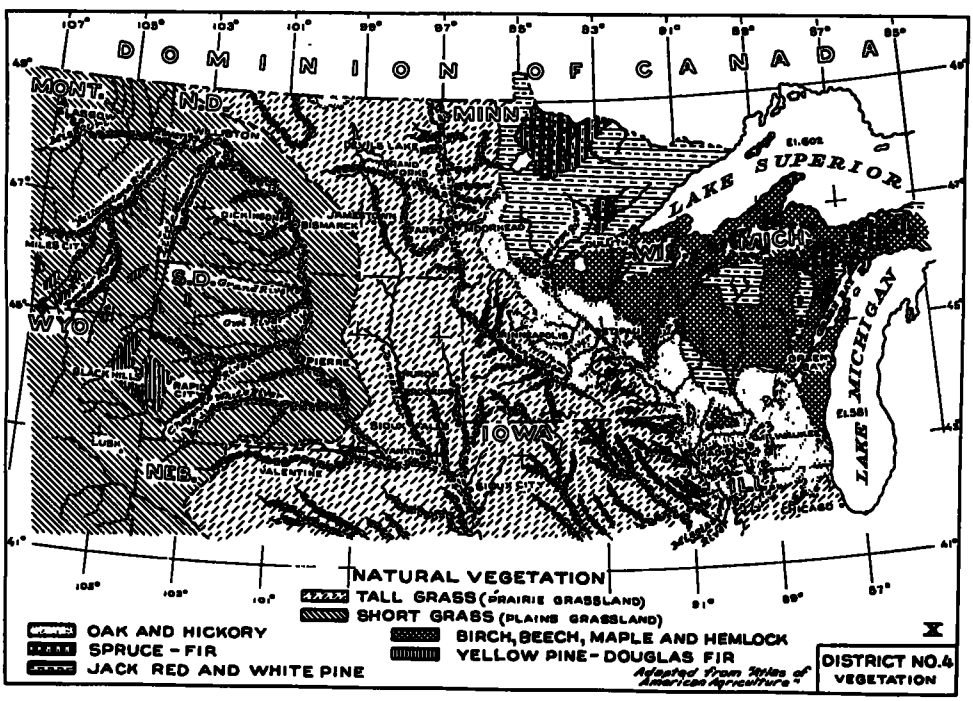


Figure 10

the highway cross-section in relation to topography, using as a basis the typical cross-sections now in regular use in each of the States of PRA District No. 4.

A summary of the typical sections is presented in Table 1, which indicates that:

(1) Graded roadbed widths vary from 40 ft. down to 24 ft. and 26 ft., although 36 ft. and 26 ft. to 28 ft. appear to be largely used, considering the region as a whole.

(2) Gutter slope ratios are 4:1 with few exceptions.

(3) The widths of gutter slopes are variable, with from 5 to 10 ft. used as a minimum. Gutter rounding is practiced in Minnesota and Wisconsin.

(4) Cut slope ratios vary from a 2:1 maximum to a 6:1 minimum, with the average about 4:1.

(5) Fill slope ratios vary from 4:1 to 2:1, with guard rail. There is indicated some relation of ratios to heights of fill, with 4:1 used on fills up to 5 ft. in North Dakota, 4:1 used on fills under 10 ft. in South Dakota, 4:1 used up to 6 ft. in Minnesota, and in Wisconsin wherever guard rail is not used.

(6) Right-of-ways are uniformly wide, ranging from 80 to 100 ft. through 150 to 200 ft.

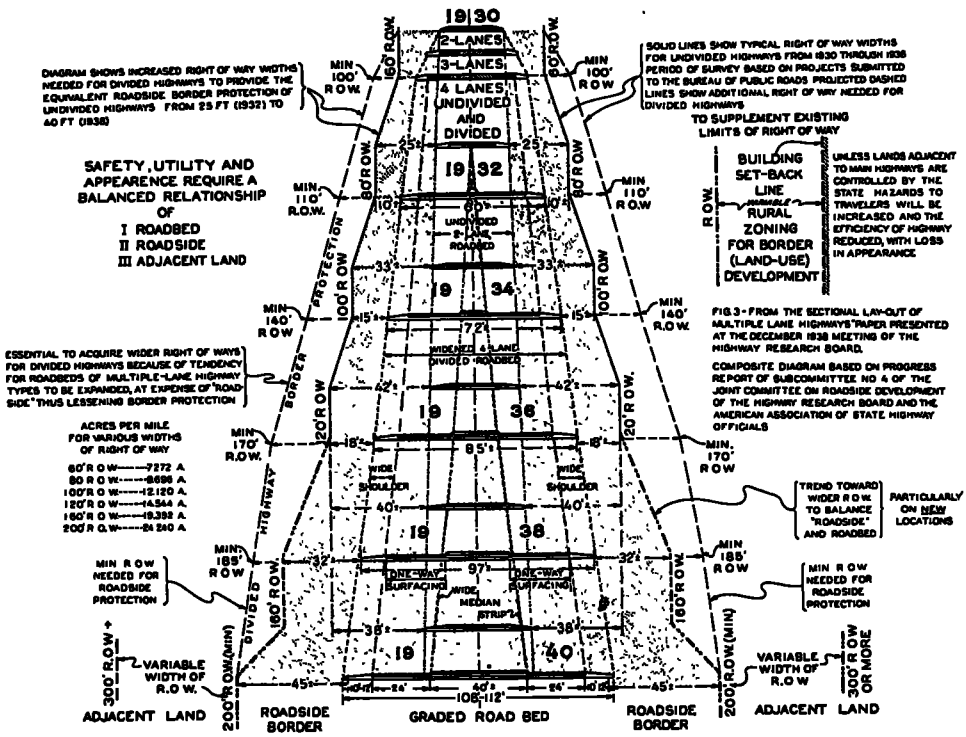


Figure 11. Trend in Expanding Widths of Divided Highways in Relation to Right of Ways

The flat and easy topography over the larger part of this region is favorable to these stream-lined cross-sections, which require adequate right-of-way width to keep fences and pole lines far enough

back from the center line so as not to cause snow drifting.

The relationship of topography to graded width as outlined in General Administrative Memorandum No. 74, issued by the Public Roads Administration under date of March 15, 1939, may be of interest.

In setting up Rules and Regulations for Secondary Highway Funds for the fiscal year 1940 and subsequent authorizations, it is stated that:

"(f) The design requirements for alinement and grade for secondary or feeder roads shall be consistent with the topography and the purpose to be served by the improvement. The minimum graded width of roadbed from out to out of shoulders shall be not less than 26 feet in easy topography; not less than 24 feet in rolling topography; and not less than 20 feet in mountainous topography with a greater width than the 20-foot minimum on through fills. Where roadbed slopes are flattened to 3 or 4 to 1 the minimum graded width in easy topography may be reduced to 24 feet. Abbreviated plans may be accepted when the

TABLE 1

COMPAPATIVE SUMMARY OF TYPICAL SECTIONS

State	(1) Graded roadbed width ft.	(2) Gutter slope ratio	(3) Gutter slope ft.	(4) Cut slope ratio	(5) Fill slope ratio	(6) R.O.W. width ft.
North Dakota..	36	4:1	10 min.....	4:1 (rounded)	Var. 4:1 to 2:1	200
	26	4:1	Var. 5 min.	4:1 (rounded)	4:1 to * 2:1	
South Dakota...	32	4:1	Var. 10 min.	5:1	4:1 (under 10′) 2:1 (10′ up) 8:1 used in borrow design	100
	26	4:1	Var. 8 min.	5:1	4:1 (under 10′) 2:1 (10′ up)	80±
Minnesota.....	40 and 36	4:1	Liberally rounded. Seeded or sodded.	4:1 2:1	4:1 (0′–6′) 3:1 (6′–12′) 2:1 (over 12′)	Variable approx. 150
	28	3:1	Rounded...	3:1	3:1	
	24	4:1	Rounded...	4:1 2:1	4:1 to 3:1	
Wisconsin.....	40	4:1 and 4:1 in combination with 10:1	Variable rounded.	Variable 2:1 max. 4:1 min. rounded.	4:1 and 2:1 (with guardrail)	100
	36	4:1 and 4:1 in combination with 10:1	Variable rounded.	Variable 2:1 max. 4:1 min. rounded.	4:1 2:1 (with guardrail)	100
	26	4:1 and 4:1 in combination with 10:1	Variable rounded.	Variable 2:1 max. 4:1 min. rounded.	4:1 2:1 (with guardrail)	66 to 80

* NOTE: For fills over 14 feet widen 1 foot and use 2 to 1 slopes.

character of the improvement warrants, provided they are in sufficient detail to show the quantity and kind of work involved."

SUMMARY

Since grading may be divided into three or four general types of topography or landscape, it is recommended that typical cross-sections be studied and developed to fit each type of topography in a proposed topographic classification for each region.

It is assumed that for smooth or easy topography the classifications "Relatively Level" and "Gently Rolling" will be uniformly understood. The moderate or rolling topography might refer to country which, while not mountainous, would require almost continuous heavy cuts and fills to meet line and grade requirements. "Light Mountainous" could be considered topography of the type generally encountered in the older and less rugged chains. The classification "Heavy Mountainous" refers to the heaviest mountain work customarily associated only with relatively limited areas in the most rugged mountains. The grading volumes per mile of highway corresponding to the several definitions may be considered roughly as doubling in quantity for each successive type in such an ordered classification; i.e., "easy topography" requiring from 25,000 to 50,000 cu. yd. of grading per mile of two-lane highway, for instance, would mean in relative terms that "moderate topography" would require about double this volume or from about 50,000 to 100,000 cu. yd., while "rough topography" would require at least four times the volume of grading for "easy topography," or from about 100,000 to 200,000 or more cu. yd.

RECOMMENDATIONS

1. It is proposed that research study develop the possibilities of classifying cross-sections on a basis of topographic relationship, showing slope ratios differentiated, for various depths of cut or fill.

2. It is desirable that typical sections show the full width of right-of-way with location of pole lines, tree planting controls and other details indicated in relation to the whole graded section.

3. It is necessary for erosion and snowdrift control that slopes be flattened and rounded as liberally as local conditions in the region permit. Highway safety, good appearance, and easy maintenance are equally served where such practices are applied.

4. Adjustments in grading for the saving of topsoil and of trees should be provided as alternate parts of the typical sections.

5. With these streamlined grading and conservation practices applied more uniformly in each region in connection with original highway construction the foundation for establishment of natural ground cover protection will have been provided at the lowest cost.

6. In order to provide for factors of safety and recreation, the modern highway should include provision for "wayside" areas in addition to the fundamentals of highway requirements.

CONCLUSION

This report in Table 1 includes typical highway sections now in regular use in one specific administrative region or highway district to show the gains which have been made in recognition of the many advantages of ordinary roadside development through integration of landscape practices in initial highway construction. The cross-sections for the region selected typify the evolutionary progress in the improvement of the right-of-way and the relation of the right-of-way to adjacent lands.

It is expected that this single regional presentation will encourage further studies along similar lines in every administrative highway region of the United States, looking toward the more uniform application of the improved practices and 6-point summary of recommendations listed.

REPORT OF SUBCOMMITTEE ON PLANT ECOLOGY

GEORGE B. GORDON, *Chairman*

Associate Landscape Architect, Public Roads Administration

THE SELECTION AND USE OF GROUND COVERS ON HIGHWAY AREAS

SYNOPSIS

This is an initial report based on the experience of the State highway departments in the *Northeast and Middle Atlantic States* in the use of low growing native shrubs, vines, and grasses as a protective cover for slopes, intersection islands, central strips of divided highways, and other areas of bare soil. *The methods listed will apply in principle wherever sod or mat-forming vegetation can be established without intensive maintenance.* The species of plants listed should of course be used only *within their natural ranges* or in localities where they have been naturalized over a period of many years.

In general the ground cover species listed and the methods of their use described are intended to apply in regions 22 to 28 inclusive as outlined on Figure 1. In all cases, however, field examination of each section of roadside, with a careful technical landscape analysis of topography, soil and existing vegetation, should be the basis for selection of ground cover plants and the methods to be used in their establishment.

One of the most essential operations in highway landscape improvement is the protection of areas of bare soil against erosion. No method is so economical or effective under average roadside conditions as the establishment of native or naturalized grasses and low shrubs or vines. The term ground cover as here used will be understood to cover annual and perennial grasses, perennial woody vines, low native colony-forming woody shrubs which do not as a rule exceed 2½ to 3 ft. in height at muturity, and certain mat- or sod-forming herbaceous plants such as the most common types of ferns.

PRELIMINARY OPERATIONS

Experience indicates that the following stages of landscape work are essential before the actual seeding, sodding or planting of ground cover begins:

Landscape Reconnaissance: It is essential that highway areas in need of protection against erosion be carefully examined and analyzed by competent technical landscape personnel. The expenditure of highway funds for seeding, sodding and planting cannot in the long run be effective except as based upon skilled analysis of soil, drainage, aspect, and other site conditions.

Preserve Trees, Topsoil, Sod, Stone: Before grading begins, provision should be made for preservation of outstanding existing trees and groups of existing shrubs or other desirable vegetation as far as this conservation can be combined with the development of a well-rounded cross section. Topsoil, sod, stone, and other native materials which can be advantageously used in landscape development should be removed from areas of excavation and stock piled if necessary. It is a serious lack of foresight not to conserve topsoil in original construction especially on new center line location in areas of cut across cultivated fields or pasture land. Topsoil is usually an inferior subgrade or fill material. Topsoil restored to the exposed surfaces of finished cuts and fills is a major factor in preventing erosion. Even without seeding or planting, native vegetation soon becomes established in topsoil covered surfaces which have been flattened to

1 on 2 or more. Slopes which already have a satisfactory cover of vegetation will as far as possible be left undisturbed during grading work. Outstanding trees, rock ledges, and similar landscape features may control or limit the extent of slope flattening and rounding.

Grading: To facilitate the rapid establishment and growth of ground depending upon the total slope height. Grading operations can usually be most economically performed as a part of original construction contracts.

Soil Preparation—Topsoiling: Following landscape grading the soil on slopes or intersection areas may be prepared for seeding, sodding or planting. Topsoil should be restored and manures

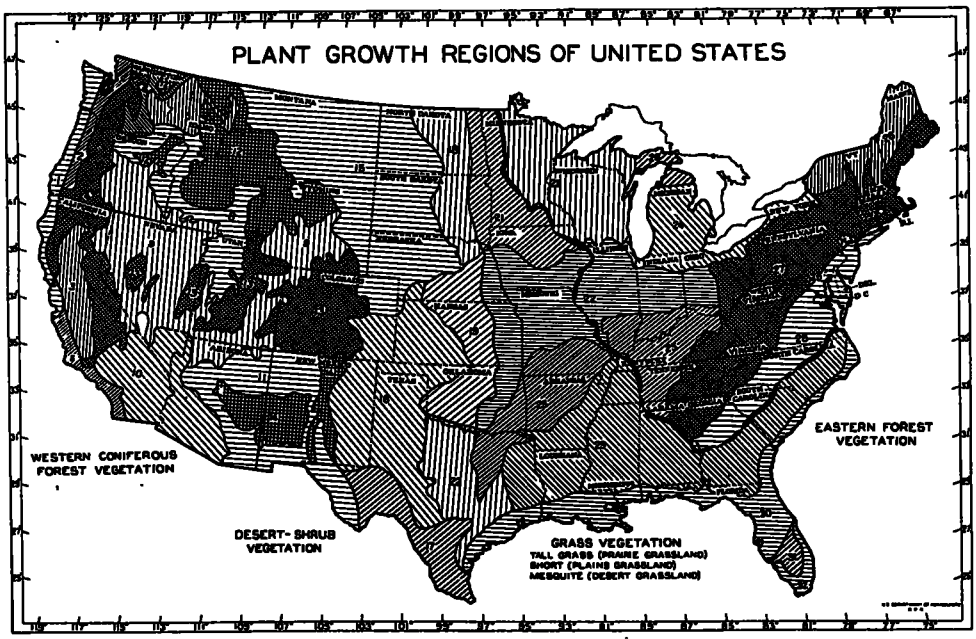


Figure 1. This Map is adapted from the Map of the United States found in figure 2 on page 4, U. S. Department of Agriculture Farmers Bulletin 1482, titled "Trees for Roadside Planting," by F. L. Mulford. Numbers within the border lines on both Maps indicate those Regions having approximately similar growing conditions for the same elevations. Darker areas are mountain or hill regions where altitude and topography have an important influence on plant growth.

covers, highway slopes should be flattened and rounded within the limits set by existing topography and available right-of-way and easements from property owners. Slopes, after being flattened, should be liberally rounded, beginning at a point between one-third and about one-half the distance between the slope stake and ditch line, and extending to a point about 5 to 15 ft. behind the slope stake toward the right-of-way line,

or fertilizers added as required by existing site conditions. For example, on low flat slopes, shoulders or intersection areas with sterile gravel or clay soils of bad texture,[1] loam or sandy loam topsoil may be broadcast to a depth of 2 to 4 in. to insure satisfactory growth of grass. Where existing soils are of fairly good

[1] For definition of soils by texture, see Atlas of American Agriculture, Part III, pages 11-12.

texture (sands, sandy loams or loams), addition of harrowed or raked-in commercial fertilizers or farm manures may be effective without the added topsoil. The amount of topsoil or fertilizer required and the method of application will be best determined by a careful examination of the site and soil by competent landscape personnel.



Figure 2. Well Rounded slopes plus a soil covering which will support plants are necessary for the rapid establishment of ground cover plants in all climatic regions.

TREATMENT OF CUT AND FILL SLOPES

No seeding, sodding or planting should be done on highway slopes until proper flattening and slope rounding as governed by existing topography, have been done.

Selection of Plants: 1. Running vines or drought resistant native colony-forming woody ground covers of types similar to such vegetation already found existing along the road should be selected for planting on high steep highway slopes. Where moderate slope (1 on 3 or flatter), good soil, and other site conditions indicate the use of seeded or sodded grass, grasses may well be used for slope protection.

2. In general native types of vines and shrub ground covers will be found available from nursery sources. Nursery grown vines, particularly, will be found most satisfactory for roadside planting.

3. Where native ground cover shrubs are available in necessary quantity near the highway, these can usually be successfully collected providing:

A. Analysis of collecting field soil indicates favorable transplanting conditions. For example, loam and clay soils favor collection; loose sands and gravels render collection difficult.

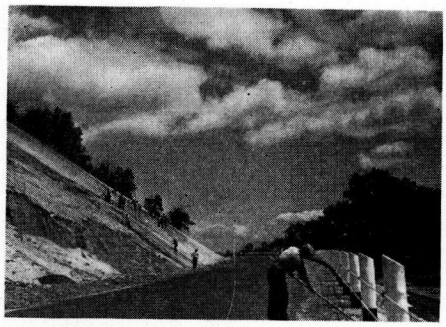


Figure 3. A Bare Sand Slope before Landscape Treatment

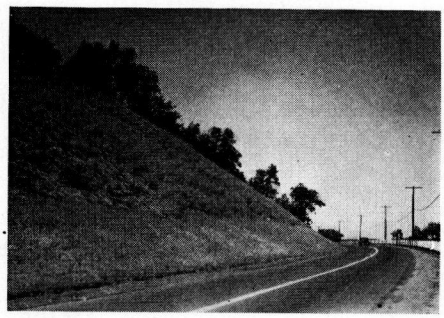


Figure 4. The Same Slope after mulching with hay, planting with sweetfern and sumac, and seeding.

B. Collection can be done during proper season. For example, most plants are best collected in spring.

C. Collected plants can be dug and handled under supervision of well-trained landscape personnel. Collection should not be attempted unless at least a "skeleton force" of experienced

labor is available. This is often the difference between success and failure.

D. Collected plants can be transplanted from the growing site directly to the planting site without delay. Heeling in of collected plants will usually mean a low percentage of survival at the planting site, and may increase costs to the point where it may be advisable to use nursery grown plants.

4. Shrub or ground cover species which do not form a compact fibrous root system after a year or two in the nursery may well be collected rather than nursery grown. This class of collectible materials includes, for example, the common sumacs (Rhus copallina and glabra), the elderberries (Sambucus species) and plants with similar coarse pulpy root systems.

Planting Operations: In general, planting operations (to be successful) will require:

1. Trained landscape personnel plus a skeleton force of experienced laborers in each untrained labor crew.

2. Particular care in inspection of nursery grown or collected plants during digging and at the point of delivery. Only healthy plants with adequate root systems should be delivered to the planting crews.

3. Care during transportation and planting to avoid drying out of root systems. Thorough watering where necessary after planting.

4. Proper mulching of planted trees, shrubs or vines after planting. In planted groups or colonies of ground cover, bare soil between plants should as a rule, be covered with two or three inches of loose straw, hay cuttings, pine needles or other available mulch. This mulch tends to conserve moisture and to keep out weeds. Rain cups must be left around individual plants or a trench planting method should be used designed to evenly distribute surface water to the plants.

5. Where mulching is not required because of good soil and abundant moisture a light seeding of rough pasture type of grasses may be done in spaces between plants on the planted slopes. For example, use 25 or 30 lb. per acre of red clover and timothy or a similar pasture seeding adapted to local soils and climate. Individual planted vines or low shrubs should always be mulched to control weed growth.

After planting, pruning of shrubs or vines will be necessary. In general, pruning will accomplish reduction of top growth in proportion to the root growth of each plant pruned away during transplanting operations. Vines and fast-growing, soft-wooded shrubs will usually be cut back almost to the ground, leaving only the first two or three buds on stems.

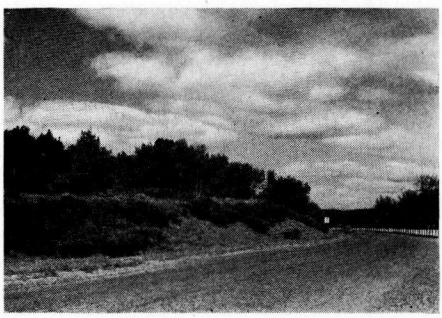


Figure 5. Planted and Natural existing sweetfern on a New England Highway. Well-rounded slopes mulched with hay or local woods litter will often cover themselves without the need for planting.

Maintenance for about the first two seasons following planting of shrub or vine ground covers will usually be necessary. For example: watering will be done regularly when needed; rain cups or trenches shall be kept in their original condition; mulch will be kept in place to adequate depth around plants; tall weeds

will be pulled up and destroyed in spaces between planted shrubs or vines; mowing of grass or weeds on areas adjacent to beds or groups of planting will be done with care, special attention being given to preventing injury to young shrub or vine growth which may come in among the grasses from seed or root suckering.

Every precaution will be taken against fire which may destroy the planting. Clean, well-mowed shoulders and shoulder slopes are the best possible fire insurance. The placing of about a half inch of loam upon dry mulch materials may also be advisable both as a precaution against fire and to prevent displacement of the mulch by wind or surface water.

TREATMENT OF INTERSECTION TRIANGLES, CIRCLES, ISLANDS, AND MEDIAN STRIPS

Intersection areas should be seeded or planted in such a manner that they merge with the adjacent backslopes and with the landscape beyond road borders. Thus in a landscape dominated by grasses where grass ground covers are properly used to cover nearby slopes, the same seeded grasses are used on intersection areas. Where, for example, planted honeysuckle or other vines are used to cover nearby slopes the same types of vines may be used on intersections as a solid continuous ground cover.

Clear Sight Distance: Planting of intersection islands should be done with low-growing plants and in such a manner that clear sight distance is maintained. Usually plants which exceed about 3 ft. in height at maturity must not be used where sight distance may be reduced by the planting of taller growing shrubs and trees. Traffic safety must be the controlling factor in all design.

Evergreen trees and high-growing evergreen shrubs should especially be used with caution at intersections. High-headed deciduous shade trees which will allow clear vision underneath their branches are the only tall plants advisable as a rule for intersection planting, in central strips of divided highways, or elsewhere where sight distance must be kept open for traffic safety.

Tall-growing flowering shrubs of ordinary "garden types" are not recommended for intersection or for slope planting on open country roadsides. Such shrubs are difficult to maintain in healthy condition on roadsides, tend to cause snow-drift formation in winter, and do not form the low, continuous, smooth mat of vegetation usually desired at intersections.

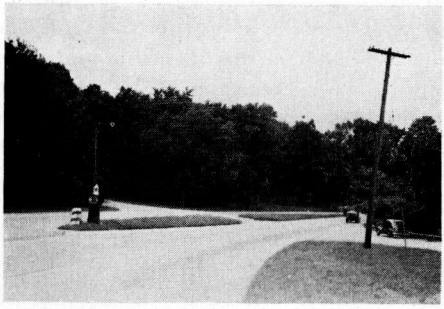


Figure 6. Honeysuckle planting on intersection islands. Planted evergreen vines like this require one mowing about every two years. By comparison grass sod is much more costly to maintain.

Planting of native low shrubs which will not exceed three feet in height at maturity, should be done in natural combinations found in adjacent fields and pastures. One type of ground cover should, however, dominate the planting in each intersection island or triangle. Solid groups of plants should cover the entire area of small triangles or should be confined to terminals and edges of large intersection areas.

Central Strips: On level intersection areas of large size native meadow types of grasses with occasional groups of high-headed shade trees located to facilitate machine mowing are usually indicated.

On level intersections of small size, or on median strips of a divided highway, solid vine or other low woody ground covers will usually be preferable to grass, and high-growing shrubs or trees should not be planted. Some exceptions to this rule as regards shade trees or flowering trees are permissible on central strips of divided highways 25 ft. wide or wider, or on slightly narrower median strips where bridges are a factor in the design. As a rule planting of trees or ground cover is not desirable on islands or central strips less than 10 ft. in width. Islands between about 6 and 8 ft. in

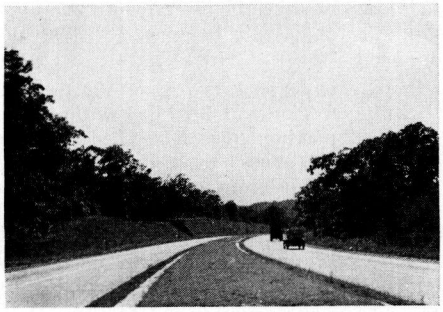


Figure 7. Median strips should be designed for easy maintenance. Here the sod can be readily mowed by modern machine methods.

width will usually best be seeded with grass. When less than 6 ft. wide, islands or central strips will best be filled with gravel or bituminous surfacing materials contrasting in color and texture with the traffic lanes, because the growth of healthy grass will under these conditions be found almost impossible over a period of years.

On rough intersections of such an irregular or rocky surface that machine mowing is impracticable, running vines can best be used as a solid ground cover using one kind of vine on the whole area. Grass ground covers will rarely be satisfactory on highway intersection or slope areas where machine mowing cannot be done readily.

LANDSCAPE TREATMENT

The experience of the States to date has led to the preparation of Tables 1-5. It will be understood that the data here given apply only to average conditions

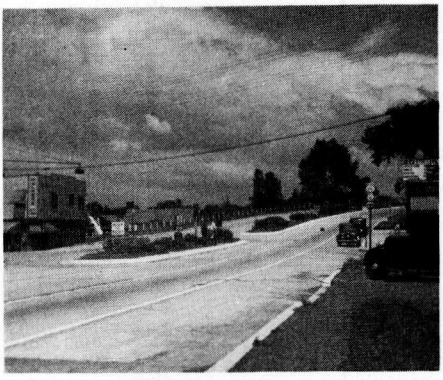


Figure 8. Median strips should be planted with low masses of solid ground cover. The groups of laurel shown here are most costly to maintain.

as listed. The methods given are intended to be flexible and must be varied as landscape analysis of each particular site indicates to be desirable. Species of plants listed are intended for use only within their natural ranges or where naturalization is no longer questionable.

These tables are subject to future revision based on the experience of the States and the Public Roads Administration.

TABLE 1

SOME RECOMMENDED SLOPE PROTECTION METHODS

For use under humid climatic conditions of the United States

Soil type	Height of slope	Slope* ratio before rounding	Recommended treatment
1. Pervious: Sands Sandy loams Sandy gravels, etc.	To 10 or 12 feet.	1 on 1½ to 1 on 2	A. Mulch with hay, straw, pine needles, etc.; plant groups or colonies of common native ground cover types. B. Broadcast manures or organic types of commercial fertilizer, mix with existing soil on slopes to average depth of 3 to 4 in.; seed with pasture grass or wild grass mixture; follow seeding with light hay, straw or peat mulch.
2.	More than 10 or 12 feet.	1 on 1½ to 1 on 2	A. Prepare large pockets, plant vines or native shrubs in irregular groups; mulch unplanted space or seed pasture grasses between plants; apply light mulch on seeding. B. On pure sands plant bayberry Indian grass, beach grass or other plants growing nearby on sandy soil. Mulch with hay or straw. (See footnote, Table 2.)
3. Impervious: Clays Clay loams Silty clay loams, etc.	To 10 or 12 feet.	1 on 1½ to 1 on 2	A. Prepare large pockets or contour trenches; fill with good loam topsoil. Plant with running vines, seed pasture grasses between plants or mulch with peat moss, straw, etc. B. Broadcast topsoil or topsoil plus equal part sand. Use harrow or rake to incorporate topsoil with existing soil to depth of 2 or 3 in.; seed with pasture grasses, follow with light ½ to 1-in. mulch.
4.	More than 10 or 12 feet.	1 on 1½ to 1 on 2	A. Treat as per method 3A above. (NOTE: Subsurface drainage may be necessary where heavy seepage occurs.) Planting in pockets or contour trenches as above.
5. Pervious or Impervious: Sands to Silty clay loams as above.	To 10 or 12 feet.	1 on 2 to 1 on 3	A. Broadcast topsoil where required because of sterile nature of existing soil; seed pasture grasses, follow with ½ to 1-in. mulch or— B. Add topsoil; seed or seed plus strip sod. Add light mulch on South or West exposures and on erodible types of soil.
6.	More than 10 or 12 feet.	1 on 2 to 1 on 3	A. Treat as per 1A, 1B or 2B above. (NOTE: Subgrade drainage as necessary.)
7. Loams Sands Clays Silts	To 10 or 12 feet.	1 on 3 or flatter	A. Broadcast topsoil; where possible, incorporate this loam with existing slope soil to depth of 3 to 4 in.; seed with lawn or pasture grasses. Mulch usually needed on erodible soils or on dry South or West exposures.
8.	More than 10 or 12 feet.	1 on 3 or flatter	A. Broadcast topsoil, strip sod, or seed and strip sod on erodible soils; mulch on erodible soils after seeding. B. Broadcast topsoil; seed; follow with light mulch on slopes facing South or West. C. Plant in contour furrows, or pocket available collected low shrubs or native vines. Seed with pasture grasses between planting, or mulch whole area of bare soil on dry slopes with light cover of well-rotted manure or hay cuttings.

TABLE 1—Continued

Soil type	Height of slope	Slope* ratio before rounding	Recommended treatment
9.	Level intersection shoulders.	Level or nearly level	A. Broadcast topsoil; pocket running vines or mass plant with a single kind of low native ground cover; mulch between planting on dry sites; seed between planting on moist soils. B. Broadcast topsoil and incorporate added soil with existing soil to depth of 3 or 4 in.; seed with lawn or pasture grasses; light mulch where required by dry soil conditions. On shoulder areas treat as per B—vines or shrubs not recommended.
10. Rock or gravel	Various slope degree and height		A. Prepare large pockets of good topsoil where digging conditions permit. Plant rapid-growing, coarse-textured vines such as wild grapes, etc. Mulch to preserve moisture in plant holes (mulch about plants 2-3 in. deep).

*NOTE: Establishment of ground covers on earth slopes steeper than 1 on 1½ is very difficult, and sometimes impossible. Where topography will not permit flattening of 1 on 1 cut slopes to 1 on 1½ before rounding, use methods 1A or 2A.

GENERAL NOTES

1. Above methods should be varied to meet existing conditions as analyzed by landscape engineer prior to landscaping work. Where a choice exists select the method of treatment which involves lowest maintenance cost in future years.
2. Study common native vegetation near highway area to be landscaped.
   a. In "grass localities"—grasses ordinarily will be favored on most bare highway areas.
   b. In "shrub localities"—shrubs and vines will be favored, particularly on steep slopes which cannot be readily mowed by motorized equipment.
3. Adopt method to site conditions—for example:
   a. On steep slopes and particularly sterile dry soil conditions, favor vine and low native shrub planting.
   b. On flatter slopes with fairly good moist soils, conditions may tend to favor grass ground cover.
   c. North and East slopes are relatively favorable to plant growth; thus less drought-resistant grasses and vines or shrubs may be used.
   d. South and West slopes, relatively unfavorable to plant growth; use drought-resistant vines and ground covers in preference to grasses. Light mulch one-half to one inch deep to follow seeding—heavier mulch two to three inches deep around and among planted vines or shrubs.
4. Where there is choice of ground cover materials, use:
   a. Plants or grasses which require least soil preparation and least cultivation or "feeding" after planting or seeding, and which are similar in type to existing vegetation along each section of highway.
5. Where grasses or planted native shrubs are used, observe general rule that natural mixtures of several kinds of grass or plants are preferable to any one kind of grass or other ground cover. Where running vines are planted, use one kind only on each planting site unit.
6. Do not plant on same planting site on open country roadsides:
   a. Native and "horticultural" types of shrubs.
   b. Fast-growing vines and erect-growing shrubs or shrub ground covers.
7. As a rule use planted native wild types of grasses, woody vines or native, low shrubs on all slopes which cannot be readily mowed by machine methods.
8. Mulch. Two types of mulch as are referred to in the table above:
   a. Heavy 2-in. to 4-in. depth of loosely compacted pine needles, straw, hay cuttings, peat moss and the like designed to prevent all weed or grass growth and to conserve moisture on the areas covered between planted vines, shrubs, or trees, and about the roots of such plants.
   b. A light ½-in. to 1-in. loose mulch of same materials as above designed to hold grass seed in place, to prevent erosion while seed is germinating, and to permit grass to grow through it.

TABLE 2

PARTIAL LIST OF GRASSES OF NORTHEAST AND MIDDLE ATLANTIC STATES

NOTE: For purposes of this report, grasses have been divided into "lawn grasses" and "pasture grasses." "Cover crops" are included in the table though they are not usually permanent ground covers and must usually be followed by, or used in conjunction with, types of true grasses for permanent ground cover protection.

"LAWN" GRASSES	REMARKS—BEST ADAPTED FOR
Bent grass—Rhode Island, etc.	Northeast coast areas—moist slightly acid soils; sand to clay
Kentucky bluegrass	Moist loams or clay loams; neutral to slight alkaline.
Canada bluegrass	Moist to dry clay loams and clays—not acid in reaction.
Red top	Various soils—not too acid or alkaline—very moist to dry.
Red or Chewing's Fescue	Sandy soils—slightly acid to slight alkaline; shade tolerant.
Rough stalked meadow grass	Fairly moist soil, as for Kentucky blue—shade tolerant.
Perennial rye grasses—Italian, English	Require fairly moist soils, rapid growth, form temporary sod; some strains are annual, not perennial. Rye grasses tend to produce a "bunchy" sod.
NATIVE OR PASTURE GRASSES[1]	
Timothy	Various good soils—not too dry or sterile.
Orchard grass	Various soils—less fertile than for timothy.
Tall meadow oat grass	Light sandy or gravelly soils—well-drained.
Meadow fescue	Forms better sod than grasses above. Moist or wet soils.
Sheeps fescue	Poor sands, clays and gravels—forms a bunchy sod.
Red top	Various soils, best if slightly acid.
Broom sedge (Andropogon)[2]	Various poor clays and clay loams—well-drained.
Indian grass (Sorghastrum)[2]	Various poor soils—well-drained gravelly or sandy clays and gravelly loams.
Beach grass (Ammophila)[2]	On pure sand, particularly on seacoast.
LEGUMES AND COVER CROPS	
White clover (native)	Good loams and clay loams—well-drained.
Red clover	Good soils—not acid in reaction.
Crimson clover	Good soils—not acid in reaction.
Alsike clover	Prefers moist lands—not acid in reaction.
Sweet clover (Melilotus)	Dry sands and clays—slightly alkaline.
Wheat, Rye, Buckwheat, Oats	Use grain crops in combination with lawn and pasture grasses—particularly as nurse crops for late fall seeding; buckwheat and oats should as a rule be sown in spring.
Hairy or winter vetch	Various well-drained sandy and gravelly soils. Sow in spring.

[1] Except for red top and meadow fescue, these are grasses which do not always form a continuous sod. Red top or meadow fescue should usually be mixed with seed formulas of bunch grasses, such as timothy or sheeps fescue.

[2] It is believed that these wild "pasture grasses" can be readily established on slopes by mowing old fields in summer and mulching bare slopes with the hay cuttings. Mowing should be done if possible just before the grasses are "ripe."

NOTE: (1) Seed of legumes if introduced on soils where they have not previously been grown, should be inoculated before sowing.

(2) Consult State Agricultural Experiment Stations for information regarding cover crops and grasses.

(3) Percentage of clovers to be sown mixed with lawn grass seeds, should not ordinarily exceed 3 to 5 percent by weight of total grass seed formula or mixture. Clovers, alfalfa, etc., should be mixed with pasture grasses as per formulas recommended by nearest agricultural experiment station.

TABLE 2—Continued

GENERAL RULES FOR SEEDING

1. Purchase and handle all species of seeds separately: mix seed formulas on the job to fit existing site condition variations.
2. Analyze variations in seeding sites. Prepare soils as required by analysis of ground conditions; usually plowing and harrowing or equivalent will be required.
3. Add topsoil where necessary; topsoil may not be required for "pasture or wild grass" types.
4. Seed only during proper spring or early fall season for each locality.
5. Under average highway conditions, seed at rate not exceeding 100 lb. of lawn grass mixture per acre (between 30 and 75 lb. usually best).
6. Rake or cover grass seeds lightly, not more than ¼-in. deep.
7. Roll after seeding—light roller not more than about 25 lb. per ft. of roller length. Do not roll heavy clay soils after seeding.
8. Mow seeded areas with scythe about time grass reaches 6 in. in height. Mow at least three times during first season to eliminate weeds.
9. Do not attempt to rely on seeding on high slopes (more than 10 or 12 ft.) with a degree of 1 on 1½ or 1 on 2. Strip sodding with seeding may succeed under favorable slope conditions, but usually irregular spot planting with native low shrubs or running vines recommended, with a light mulch or seeding between planted areas or individual plants.
10. As a rule, solid sodding is recommended only as a last resort because—
    a. Sodding has the highest cost per square yard of slope surface treatment of the above methods, and involves a high maintenance cost per unit of area in future years.
    b. Because sod may be established temporarily on almost any degree of slope, there is a tendency to overuse this method. Satisfactory grass sod can seldom be maintained in good condition on steep dry slopes over more than a few years, without extra care and expense.
    c. Solid sodding on highway sections where grass is not the predominant type of existing ground cover, tends to retard the establishment of native low shrub ground covers which are more permanent and require less maintenance (in the long run) than grass sod.
11. Observation of existing grasses in a locality is the recommended basis for selection of grass species to be sodded or seeded.

TABLE 3

PARTIAL LIST OF NATIVE AND NATURALIZED VINES

Recommended for Trial on Highway Slopes in North and Atlantic States

Average Spacing
1. 3 to 5 feet
2. 5 to 8 feet

Adaptation to Soil and Shade
D—dry—well-drained
M—moist—well-drained
W—wet—poor drainage
S—will stand some shade

Common name	Latin name	Symbol
Boston ivy	Ampelopsis tricuspidata (suburban areas only)	1-D to M
Porcelain Amp	Ampelopsis heterophylla	1-M
Monkshood Vine	Ampelopsis aconitifolia	1-M
Virginia Creeper	Ampelopsis quinquefolia	1-M-S
Actinidia	Actinidia arguta	2-M-S
Trumpet Creeper	Bignonia radicans and B. capreolata	2-D to M
Bittersweet	Celastrus scandens	1-D to M
Oriental Bittersweet	Celastrus orbiculatus	1-D to M
Virgin's Bower	Clematis virginiana	1-W to M
Sweet Autumn Clematis	Clematis paniculata	1-W to M
Dutchman's Pipe	Aristolochia sipho	2-M-S
	Aristolochia macrophylla	2-M-S
Matrimony Vine	Lyceum chinense	1-D
	Lyceum halimifolium	1-D
Hall's honeysuckle	Lonicera japonica halliana	1-D-S
Dutch woodbine	L. periclymenum belgica	1-D to M
Trumpet honeysuckle	L. sempervirens	2-D to M
Grape honeysuckle	L. prolifera	1-D to M
Other climbing honeysuckles	Lonicera species	
Wichuraiana rose	Rosa wichuraiana (many hybrid climbers)	1-M
Kudzu vine	Pueraria thunbergiana (Virginia and south only)	1-M
Wild grape	Vitis cordifolia and other local species	2-D-S
	V. rotundifolia—V. labrusca, V. vulpina	2-D-S
	V. aestivalis and other species	2-D-S
Wisterias	Wisteria chinensis (suburban areas only)	2-M
English ivy	Hedera (suburban areas only)	1-D to M

NOTE: Vines normally should be nursery-grown though they can be successfully collected where commonly occurring along roadsides on heavy sod, as does Hall's honeysuckle.

All plants should be planted in large pockets or trenches of good topsoil where existing soils on slopes are poor.

Prior to planting, all highway slopes should be examined to determine:

1. Location of areas of good soils above dry cut slopes. Trailing vines planted here will cover the slopes below.
2. Location of seepage areas on the face of dry slopes where vines can be readily established.
3. Location of points at the foot of slopes where the "water table" is near the surface.

NOTE: Do not plant rapid-growing vines on same slope area with erect growing or flowering shrub or herbaceous ground covers which may be overgrown by vines. Mulch spaces between planted vines with hay, straw, pine needles or other available materials to control weeds, or seed open spaces on slopes with fast germinating grasses adapted to soil conditions where soil conditions are favorable to plant gowth.

Warning: Certain vines such as Hall's honeysuckle and oriental bittersweet are not frost-resistant in northern Maine west to Minnesota.

TABLE 4

PARTIAL LIST OF NATIVE AND NATURALIZED SHRUB GROUND COVERS
For Highway Slopes in North and Atlantic States

Plants Marked:

C—have been collected
N—commonly available nursery grown
W—require moist or wet sites
D—will grow on dry sites
S—will tolerate shade

Average Spacing Recommended indicated by:

1. 1 to 2 feet
2. 2 to 4 feet
3. 3 to 5 feet

Common name	Latin name	Symbol	Specially adapted to soil
Jersey tea	Ceanothus americana	C-2D	Dry, sandy slopes
Hazelnut	Corylus americana	N-3DS	Moist woods edges
Beaked hazelnut	Corylus rostrata	N-3DS	Moist woods edges
Sweet Fern	Comptonia aspleniifolia	C-1D	Sands and gravels
Scotch Broom	Cytisus scoparius	N-3D	Moist cool soils
Dwarf Bush Honeysuckle	Diervilla trifida	C-N-2W	Moist acid soils
Bayberry	Myrica carolinensis	N-2D	Sands and sandy loams
Wax Myrtle	Myrica cerifera	N-3D	Sands and sandy loams
Male Berry	Lyonia ligustrina	C-3W	Moist acid soils
Stagger bush	Lyonia mariana	C-3W	Moist acid soils
Fragrant Sumac	Rhus canadensis	N-2D	Well-drained soils
Smooth Sumac	R. glabra	C-1D	Well-drained soils
Dwarf Sumac	R. copallina	C-1D	Well-drained soils
Meadow Rose	Rosa blanda	N-C-3D	Moist sandy soils
Swamp Rose	R. palustris	N-C-3W	Wet soils
Sweetbrier Rose	R. rubiginosa	N-C-3D	Moist sandy soils, clays
Rugosa Roses	R. rugosa (many hybrids)	N-3D	Moist sandy soils
Prairie Rose	R. setigera	N-C-3D	Moist clay soils
Wild Roses	Various local species	C-3D	Well-drained soils
Northern Dewberry	Rubus procumbens	C-1D	Slightly acid soils
Southern Dewberry	Rubus trivialis	C-1D	Acid to neutral soils
Meadow Sweet	Spirea alba	C-2W	Wet soils—sand and loam
Hard-hack	Spirea tomentosa	C-2W	Wet soils—sand and loam
Snowberry	Symphoricarpos racemosus	N-C-3D	Various soils
Coralberry	Symphoricarpos vulgaris	N-C-3D	Various soils
True Ferns (collected sods)	Many species as adapted to local conditions	C-1D to W-S	Moist shaded areas
Bracken Fern (collected sods)	Pteridium, in oak-pine regions	C-1D	Acid sands and loams
Low Blueberry and Huckleberry	Vaccinium and Gaylussacia —many species	C-2D	Acid—well drained
Bearberry	Arctostaphylos-uva-ursi	N-C-1D	Acid—sands
Mountain Laurel	Kalmia latifolis	N-C-2W-S	Cold north and east slopes and under pine and oak woods
Lambkill (collect in sods)	Kalmia angustifolia	C-2D-S-W	Cold acid soils
Bunchberry (collect in sods)	Cornus canadensis	C-1W-S	On acid soils
Periwinkle	Vinca minor	N-1W-S	Suburban roadsides

NOTE: The plants listed above should be used *when they are common* on nearby borders of highway slopes to be protected from erosion. They can be collected in sods if not available from nursery sources.

These plants should be used in extended colonies or drifts in lots of 100 to several thousand in one group or colony. Plant in same type of soil and on same exposure as where found in nearby areas,

TABLE 5

Preliminary List of Trees, Shrubs and Vines for North and Middle Atlantic States

Includes a number of native and naturalized trees, shrubs and vines well adapted to Roadside Landscape Use in Northeast and Middle Atlantic States. (With special reference to spacing, methods of arrangement and relationships between various types of plants.)

Note: The following list includes certain typical combinations of species of native plant materials which commonly exist on open country roadsides and are available from nursery sources or may be "nursery collected" from the wild. The list is prepared mainly for the purpose of illustration of certain principles of recommended landscape practice and will be subject to future addition and modification. The general suggestions which follow will apply particularly to open country roadside projects. On city or suburban roadsides, they may have little or no bearing in some cases.

Legend:

N—Best nursery grown C—Collectible
C S—Transplant in spring C F—Transplant in fall
B & B—Should be balled and burlapped

LARGE EVERGREENS

Large evergreen trees are not considered as a substitute under usual roadside conditions for the large deciduous trees which are more valuable for shade. Evergreens are normally of value for screen purposes—and will normally be used only where mixed stands of hardwoods and evergreen or pure evergreen woodlands occur along certain roadsides. Spacing becomes larger with larger planting stock within the limits stated.

Tree Pines	Recommended average spacing ft.	Remarks Special character Pines
Pinus strobus.....N—B & B	12 to 20	Plant pines in irregular groups, not in squares or straight lines.
Pinus resinosa .....N—B & B	12 to 20	Use against background of sky or hillside, whenever possible in mixed sizes.
Pinus rigida..... N—B & B	12 to 20	Do not use where smoke and gas from freight yards and factories is a problem.
Pinus virginiana.... .. ..C—B & B	12 to 20	Plant pines with front and filler planting of common native shrubs and small trees adapted to sunny dry sites: Combinations: With birches, common sumac species, flowering dogwood, cercis, amelanchier, hawthorns etc.
Hemlocks.....N—B & B	10 to 30	Plant in massed groups of rounded outline in good moist soils.
White Spruce.....N or C S, B & B (Use white spruce, P. canadensis, in northern States where common in native landscape.)	10 to 30	Adapted to under planting under high deciduous trees. Use in combination with and background for—white birches, Kalmia and Rhododendron, azalea, etc., flowering dogwood, cercis, amelanchier, etc.

TABLE 5—Continued

Tree		Recommended average spacing ft.	Remarks Special character
Junipers..... ..... Juniperus virginiana Juniperus communis	N—B & B	6 to 15	Plant in irregular scattered groups in dry sunny sites—never use for under planting. Use in combination with—White birches—high bush and low blue berries. Ground juniper and blueberries. Common sumacs, hawthorns, etc. Bayberry, sweet fern, etc.
Arbor vitae.....	N—B & B	6 to 15	Plant in rounded dense groups—on moist sites—can be used for under planting.

Arbor vitae is not recommended for planting on roadside except in Northern New York, Vermont, Massachusetts and North in regions where arbor vitae is normal part of landscape—use with larch along streams, swamp edges, etc., to emphasize topographic changes.

GENERAL NOTE: All evergreens best transplanted in early spring or early fall. Plant evergreens on open country roadside only where each type of tree used is prominent among existing growth along the road or where a winter screen is needed.

LARGE DECIDUOUS TREES

The large shade trees are the backbone of all good roadside planting. Each region and each type of topography within each region has certain existing shade trees which form a basis for selection. Always merge types of tree planting into each other without abruptly stopping one type of planting to begin another. Do not plant large shade trees under public utility wires.

Ridge Types			
Oaks (except willow and pin oak).....	N	25 to 60 or 75	Plant on hillsides, exposed sunny sites where soil is adapted to particular species used. Use as individuals or in loose irregular widespaced groups; where possible use one kind of tree as most important feature in each topographic unit.
Sugar maple.....	N		
Red maple.....	N		
Honeylocust..	N		
Intermediate or Bottom Land Types			
Oaks (as above).....	N	25 to 60 or 75	Plant as above.
Maples (as above).....	N		
Honeylocust.....	N		
Elms.	N	35 to 60 or 75	Do not use elms in masses—Plant in wide spaced groups to allow full top development. Warning: extensive elm planting will require annual spray program to keep trees in healthy condition. Use elm mainly in association with other species.
Tulip poplar.....	N		
Beech.	N	35 to 60 or 75	
White ash.....	N	35 to 60 or 75	
Lowland and Shore Types			
London plane.....	N	30 to 40	In groups or drained sites.
Pin and Willow Oaks.....	N	25 to 50 or 60	Plant in connected groups on moist drained sites.
Sweet Gum.....	N	20 to 40	On drained moist sites.
Larger willows.....	N	20 to 40	Plant on wet sites where other species will not grow—to frame water views, avoid weeping willow types in open country.
Tupelo.....	N	20 to 50	
Red Maple.....	N	as 20 to 50	

NOTE: Avoid use of willows in Northeast where the willow blight is prevalent.

TABLE 5—Continued

Tree		Recommended average spacing ft.	Remarks Special character
Poplars—			
P. balsamifera P. tremuloides P. eugenei		15 to 40	Plant in relation to existing poplars of some species—Use for quick screen only. Not recommended for general roadside planting. Never good street trees.
Silver maple			
Canoe and river birch	N—C S	12 to 30	Plant along shores to frame views.

Birches can be collected to ⅜ in. caliper during 2 or 3 weeks in spring when buds are opening—Birches should not be transplanted in fall.

SMALL AND FLOWERING TREES

The small and flowering deciduous trees usually are of secondary value to tall shade trees in roadside planting. They are mainly to be used to front down existing tall-growing trees, to accent views—and structures—and in other special cases. The smaller deciduous trees may be planted, however, under wires or on narrow right-of-way or otherwise where there are limitations to height and spread of planted trees.

Flowering trees should be used in connection with low ground covered or grass-covered areas, should not be mixed with high-growing shrubs unless spaced to prevent crowding by surrounding growth. Japanese and Chinese flowering cherries not well adapted to roadside planting. All flowering trees best transplanted in spring.

The flowering or small trees have the proper main purposes of tying in the roadside to the adjacent landscape. They are used, as may be the flowering trees, in an intermediate position between tall trees in the background and lower shrubs, ground cover or grass along the edge of the roadbed.

The Hawthorns			
C. cordata	N—B & B	7 to 12	Naturally appearing groups.
C. mollis C. crusgalli C. punctata, etc.			Plant in connected dense irregular groups—to merge with existing materials of the same or related species. Dry, sunny locations with preferred clay soils. Good thorny hedge plant or screen adapted to meadow country.
The Wild Crab Apples			
Malus coronoria	N—B & B	12 to 20	Plant in specimens or in loose irregular groups to allow sunlight on all sides of tree (dwarf crab like M. Sargenta spacing 6-10 feet). Use in open meadow or orchard country roadside.
Malus ioensis Malus sieboldi			
Wild Plums			
Prunus maritima	N—B & B	6 to 12	Plums and crabapples best handled B & B.
Prunus americana			
Flowering Dogwood			
Cornus florida	N—B & B	8 to 20	Plant as specimen or in masses or groups to front down existing woodland, or in studied relationship to existing high trees. Half shaded areas with preferably rich soil.

Flowering dogwood is an excellent small shade tree under wires where tall-growing trees cannot be grown.

Red Bud			
Cercis canadensis	N—B & B	8 to 12	Use with and like above in groups of 5 to 7 or more. To be effective, groups should be larger than associated dogwoods.

TABLE 5—Continued

Tree		Recommended average spacing ft.	Remarks Special character
June Berry or Shad Bush	N—B & B	7 to 12	Use along streams or swamp edges, planting in scattered irregular groups with background of hills or other trees.
Blue Beech	N—B & B	7 to 12	
Grey Birch	N—C S	10 to 15	Plant on sites as found in nature. Birch in open meadow lands in full sunglight on variety of soils. Hornbeam, silverbell, and hop tree along edge of woodland in moist good soil and for underplanting under tall shade trees.
Hornbeam	N		
Hop Tree	N		
Silverbell	N (2 species)	12 to 20	

DECIDUOUS AND EVERGREEN SHRUBS

The taller shrubs may be used to prevent erosion on slopes as are the vines and ground-cover plants. They are as a rule in most open-country roadsides the least important class of plant material, and may often be omitted from the planting in open cultivated types of country because the vines and low ground covers are usually more effective in preventing erosion than the higher growing shrubs. The shrubs have their most important function in the tying in of the planted roadside with the adjoining open country landscape. There are two important types of native shrubs from the standpoint of arrangement—(1) Shrubs which naturally occur as specimen plants or in wide spaced thickets, (2) Shrubs which naturally grow in colonies or dense rounded masses closely spaced.

(1) The following group of shrubs will normally be planted with a wide spacing based upon the width of plants at maturity. They may be planted for various purposes to be stated on plan sheets, but their use will usually be based upon the occurrence of the same species on or near the roadside to be planted. These types of shrubs will be planted as specimens or as small groups for accent purposes. Not suitable for ground cover uses as are group 2 following:

The Viburnums			
V. dentatum	N—C S	4 to 7 depending on species used	Plant in connected loose groups to front down existing or planted trees. Can be used under shade for screen or as a "filler."
V. cassinoides	N—C S		
V. lantana	N—C S		
V. lentago	N—C S		
V. prunifolium	N—C S		

The Viburnums are valuable for flowering effect and fall color on moist sites.

High Bush Blueberry	N	5 to 7	Plant as specimens or in loose scattered connected groups.

Exceptional fall color—use in combination with junipers and low blueberry types, with deciduous holly, alder, or other natural associates.

Sambucus in 2 varieties (Elder)	N—C S	3 to 7	Plant on moist sites, open sun, stream borders, etc. and at culverts and bridges.
Ilex verticillata (Holly)	N—C S	5 to 7	Use both staminate and pistillate forms of holly.
Rhus typhina	C S	4 to 7	Plant in loose irregular rounded groups for screen purposes or to front down woodlands with naturally associated plants in preferred types of soils.
Rhododendron species	N		
Nine Bark	N		
Hazel	N		
Witch Hazel	N		
Bush Dogwoods (except C. paniculata)	N		

TABLE 5—Continued

Tree		Recommended average spacing ft.	Remarks Special character
Alder	C S		On moist sites.
Wild Roses (Rosa setigera, R. multiflora and other taller growing types).	C S or C F	3 to 5	On well drained drier sites.

This group might be expanded almost indefinitely to include (on suburban roadsides only) the common types of garden shrubs, forsythias, deutzias, bush honeysuckle, mock orange, lilacs, etc., many of which may be used (in landscapes where their types already occur) in the same way as flowering trees. These high-growing shrubs may be used behind and among groups of ground covers (see class 2) for variety and ornamental effect. Ordinarily native and garden types of shrubs should not be combined in the same planting.

SPECIAL NOTE: Do not plant tall-growing shrubs in central strips of divided highways or at intersections where clear vision may be interfered with.

(2) The following shrubs of both flowering and non-flowering (inconspicuously flowering) types are used as ground covers and should be planted in dense colonies of rounded irregular outline. Because of their natural close spacing they will be normally used in hundreds or even thousands. These ground covers can be most effective when one or two kinds tend to dominate the planting on one-half mile or more of roadside.

Sumacs			
Rhus glabra, Rhus copallina, Rhus canadensis	C S	18″ to 3′	Plant in dense masses or colonies, in drifts, extending over slopes. Usually 100 to several thousand in each continuous drift or colony.
Low Bush Blueberry, Huckleberry	C S (sods)	18″ to 3′	
Cornus paniculata	N C S	2 to 4	
Azaleas	N	2 to 4	Plant to extend same types of existing growth on various soils and in full sunlight. Azaleas and Kalmias usually best on fairly moist acid soils in part shade. Sweet fern and bayberry do well on sandy soils.
A. nudiflora	N		
A. calandulacea, etc.	N		
Sweet Fern	C S		
Bayberry	N—C S		
Rhodora	C S (sods)		
Wild Roses (low growing)			
Rosa blanda	N—C S or C F	2 to 4	On dry, well-drained sites, in extended drifts or colonies.
Rosa palustris	N—C S or C F		
Rosa humilis	N—C S or C F		
Laurel			
Kalmia latifolia	N or C S B & B	3 to 4	Do not plant on dry sterile slopes.
Kalmia angustifolia	C S	18″ to 3′	Best on acid soils at edge of pine, oak woods.
Juniperus communis	N—C S B & B	3 to 7	On well-drained sites.

Many other native ground covers of this type will be found in each locality. In practice the commoner, easily collected shrubs, such as sumacs and wild roses will usually be used for many hundreds of feet along roadsides to cover slopes and to tie in the roadside with existing growth of the same type as that planted. Laurel in certain regions may be the commonest existing ground cover and in such regions will tend to be used in roadside planting to the exclusion of other types of ground cover. Except for wild roses native ground covers should be collected in spring—not fall season. Rhododendrons, kalmias and azaleas require slightly acid soils.

VINES

The vines are the best of all ground covers since they tend to form a dense mat of root and top growth in a minimum of time. Vines may be divided into two rough classes, (1) Strong-growing rampant types, (2) Slow-growing types. All vines should be nursery grown unless otherwise stated.

TABLE 5—Continued

Tree	Recommended average spacing ft.	Remarks Special character

(1) The following partial list of vines covers certain types valuable for roadside planting. Most of these vines (if heavy rooted 2- to 3-year old plants can be secured) can be spaced 4′ to 7′ apart. In general, vines should be planted in good topsoil, larger trenches or pockets being desirable on hot sunny slopes with poor soils than on partly shaded slopes with good soils. It will be understood that slope flattening and rounding must always precede all planting, particularly of ground covers.

The Wild Grapes Vitis cordifolia Vitis labrusca Vitis vulpina Vitis rupestris, etc.	5 to 8	Adapted for use on high, rough, rocky slopes where large pockets can be dug for topsoil. Do not use rapid-growing vines on same planting site with shrubs or flowering trees.
Actinidia arguta	5 to 8	
Honeysuckle Lonicera japonica and other species	4 to 6	Best adapted to clay and loam soils, and on north and east slopes. In clays, sands and gravels become established on sunny south or west slopes very slowly. Covers bare soil rapidly after first season. This vine ideal ground cover on islands. A very strong grower. Do not plant honeysuckle in association with shrubs or low trees. Not hardy north of Massachusetts along coast.
Bittersweet 2 species Celastrus scandens Celastrus orbiculatus	4 to 5	Bittersweets tend to grow erect for first few seasons before spreading over ground. Usually slow growing first two years after planting. Best planted on partly shaded slopes, but will stand heat and drought after becoming established.
Clematis C. virginiana C. paniculata Akebia quinata	4 to 7	Best planted on north and east slopes and on fairly moist locations. Difficult to establish on hot, dry slopes. Requires large pockets of good topsoil for good flowering effect.
Virginia Creeper, etc. Ampelopsis quinquefolia Ampelopsis aconitifolia Ampelopsis heterophylla Bignonia radicans Lyceum chinense	3 to 7	Good rapid-growing vines. Vigorous climbers, some with good fall color. Tend to grow rapidly in length but some species do not always form dense wide mats as rapidly as honeysuckle or wild grape.

(2) Slow-growing vines. This group of vines is not so well adapted to roadside use as the above. This slow growth may have definite advantages in gound cover planting in small traffic islands, or in planting vines on stonework, bridge, or culvert parapets and headwalls, etc., where it is not desirable to completely hide stonework.

Wichuraiana Rose	4 to 6	A low, flat-growing vine which trails rather than climbs. Adapted to fairly rich soils on slopes which are not too hot or dry. Excellent ground cover on traffic islands.

Common hybrid climbing roses are not usually recommended on open country roadsides except near homes and gardens.

TABLE 5—Continued

Tree	Recommended average spacing ft.	Remarks Special character
English Ivy (hedera helix).....	2 to 4	Slow-growing evergreen vines which climb stonework wall. Ampelopsis species slow to grow compared to other of genus. Adapted for use on bridges and retaining walls.
Euonymus—radicans..... and other species..... ...		
Boston Ivy Ampelopsis tricuspidata		
Geranium creeper A. tricuspidata lowii		
Periwinkle Vinca .....N or C S	18″ to 2′	Slow-growing though fine flowering evergreen ground covers. Trailers rather than climbers.
Bearberry Arctostaphylos .....C S or N	1′ to 18″	Bunch berry is a small shrub rather than a vine. This group all are slow-growing evergreen ground covers. If available in good field collected sods, first two may be used for roadside erosion control on limited areas.
Bunchberry Cornus canadensis		
Partridgeberry Mitchella repens		

The bearberry, bunchberry and partridgeberry are of very refined character. Not adapted for use on ordinary roadside conditions. Can only be recommended on limited sites, i.e., as a ground cover on traffic islands, near monuments or in other special situations of relatively small area.

1. General Note: As a rule ground cover planting cannot readily be established except on well-rounded slopes 1 on 1½ or flatter. On dry sterile slopes mulch between plantings serves to conserve moisture and to control weeds.

2. As a rule grass seeding or sodding should be done mainly on roadside areas which can be readily mowed. Since machine mowing cannot be done on slopes steeper than 1 on 3, or 1 on 4, ground covers are indicated wherever slopes are higher than 5 to 10 feet and steeper than 1 on 3.

3. Important Note: Use low colony-forming shrubs or vines on islands of divided highways and intersection triangles to replace grass. On steep slopes use hay, straw or other mulch between planted vine or shrub individual plants or groups, or seed pasture grasses or cover crops in space between planting.

LOCATION AND ROAD-FOCUS

By Jac L. Gubbels
Texas State Highway Department

SYNOPSIS

A new term, "road-focus," has been coined to define the focal point at which eye strain so frequently develops, resulting in confusion and lack of alertness on the part of the driver. At the lower speeds the eyes are relaxed and able to see objects well off the side of the highway, as well as the road ahead. As speed increases, this range of focus narrows until side view entirely disappears. Such concentration of the eyes results in eye strain and increases the likelihood of accidents. The author has noted that on long monotonous stretches of road the eye strain is relieved by means of group plantings, such as groves of trees in bottle-neck fashion, which narrow the horizon and give the eye vertical objects to rest on, thus softening the focus.

In general, such measures have thus far been corrective, not preventive. Proper thought in location can prevent eye strain by making the road blend into the landscape. Location lines can be directed towards such immovable objects as tree groves or church steeples, and then curved easily around such prominent features. Locations of this type will bring prominent features of the landscape directly into the line of vision of the driver, making it unnecessary to take his eyes from the road ahead.

The program of roadside development has brought about innumerable problems. Roadside development properly begins with location; it is inseparable from construction, and then continues on with maintenance. Roadside development should not be dealt with as something apart in the process of road construction. It is too intricate a part of all features in road design.

For several years a large problem has been the reconstruction of old roads. Following a transition period a broader viewpoint has been taken as greater highway problems presented themselves. With the more speedy vehicles, the roads simply had to be designed to fit the automobile. In many instances, instead of a complete revolution in design, often a mixed design was utilized in which the old was coordinated with the new practices, and roadside development was used to camouflage the defects. In reality, roadside development should be the leader in the field of advancing new thought in every phase of highway construction, particularly that of location. Let us then view how important location is in roadside development.

I have coined the term "road focus." Road focus is that focal point where eye strain so frequently develops, resulting in confusion and a lack of alertness on the part of the driver. During the transition from the old to the new, quite a few long tangent highways were constructed. These long tangent highways are of no particular danger to the slower moving automobiles. The eyes at low speed are not focussed at any particular given point; they are relaxed and able to absorb not only the objects on the highway proper, but also the fields beyond the right-of-way. As speed is increased, the range of focus narrows to where side views disappear. In other words, the action of the eyes may be likened to the rays of an adjustable flashlight. At a near point the instrument is adjusted to throw a fanlike spray of light which illuminates a broad area, but at a far point it must make a narrow beam which brings a distant spot into view, but leaves all else in semi-darkness. This concentration of the eyes results in eye strain, and as a result, side-swiping, head-on collisions, and other curious accidents occur. On the old, monotonous roads

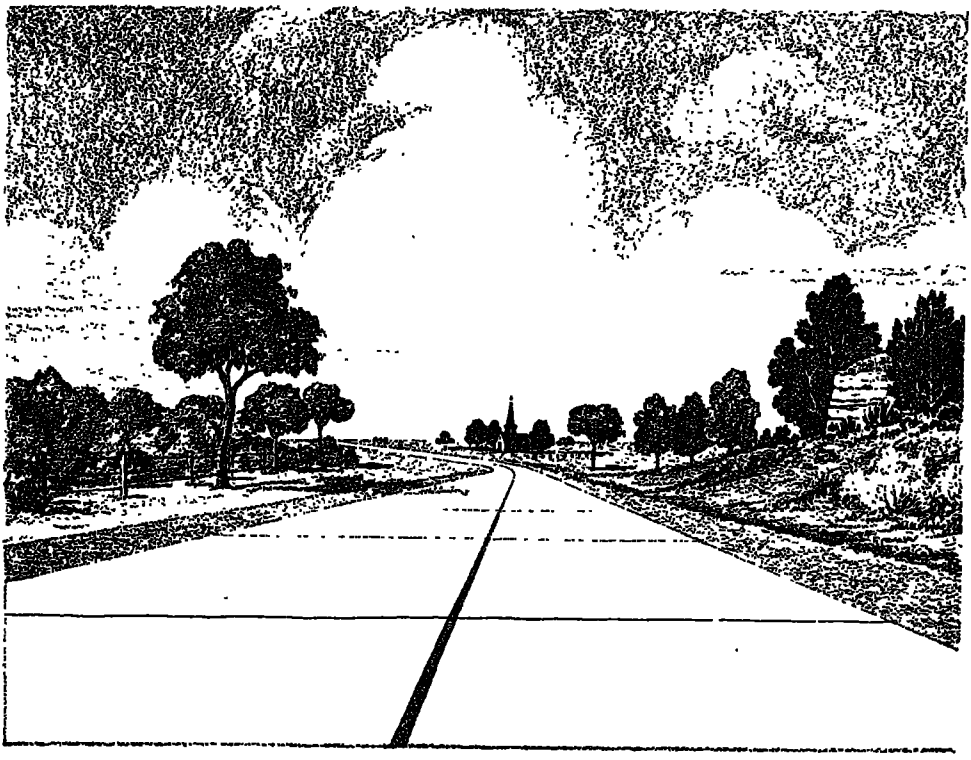


Figure 1. The Driver's Attention is Directed to the Highway with the Steeple in the Distance

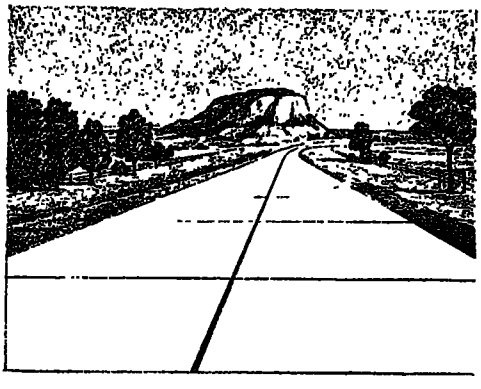


Figure 2. The Direct Approach to this Prominent Landscape Feature Presents an Interesting Picture of Light and Shade.

which will not be reconstructed for many years, we try to remedy this condition by means of group planting to narrow the horizon, which give the eye vertical objects to rest upon, softening the road focus, and preventing eye strain. This method of reducing eye strain works, as I have often observed among fellow drivers. Groves of trees in bottle-neck fashion are invaluable to shorten the focus and prevent eye strain. However, the remedy must be understood as artificial, because it is a corrective measure rather than a preventative one. Thought in location can prevent eye strain; if not, then it may be termed a wrong location, regardless of its destination or objective.

In the past, it was not unusual for the engineer to direct the line at removable objects—a tree, or an old barn, for example. These objects could be removed. This gave the engineer an opportunity to construct the road on the most direct route, with variations from a straight line reduced to a minimum. What would

happen if the location engineer directed his instrument at an immovable obstruction, such as a hill, a dam, a grove of trees, a country church, a rural home, the vertical architectural features of a village, or any one of the innumerable features which can be found in the landscape? Such a highway disappears naturally into the landscape while at the same time permitting the driver to view the scene before him. In other words, if the approaching highway leads directly towards a grove of trees, or other obstruction, and then curves easily to the right, or to the left away from the grove of trees, the landscape becomes the prominent feature.



Figure 3. On Construction these Trees were Preserved in a Bottle-Neck Design

Location is, and will continue to be a technical engineering problem. However, since a road is laid through a landscape, it is the landscape with which we are concerned in order to reveal greater beauty, insure greater safety, more economy, and present to the motorist and the engineer wider opportunities for an appreciation of the landscape.

From an engineering point of view, the highway means many things. To one intimately familiar with the problems of construction and maintenance, almost any highway is interesting. To the engineer, a road is much like reading an interesting book. He occupies himself with the profile, the vertical and horizontal curvatures, sight distance, passing distances, cross-sections, erosion, drainage structures, shoulders, pavement, center striping, sign marking, bridges, grade separations, intersections, and other innumerable items. These features present to him a continuous variety of interest which causes him to maintain his alertness while driving. However, these features are of such a technical nature that they do not concern the casual driver. Therefore, to the average motorist the road is less interesting than it is to those who deal with highway construction.

At low speed it was natural for the motorist to take in the scenic views which lay on either side of the right-of-way. However, since speed has increased, it becomes more and more apparent that those side vistas, which were once enjoyed, are now usually lost. Should a driver take his eyes from the road ahead, he endangers his life and the lives of others. It follows then that in order to make modern motoring more interesting, the scenic vistas should be within the motorist's vision ahead of the car, instead of merely a straight highway constantly in front of him.

In any given terrain it is possible to make use of some extraordinary features in the landscape to add variety to the road. For the purpose of safety, the scenic beauty of a highway can often be improved by even a slight deviation from the usual directness. After all, directness has no advantage other than to provide the shortest distance possible. This shortest distance often necessitates high construction costs, and excessive maintenance expense.

It is important that road focus be given more study. The underlying reasons for road focus will undoubtedly receive more attention by those who build highways for the traveling public, as well as by the automotive industry. Though the application may be subject to corrections which will come from experiments,

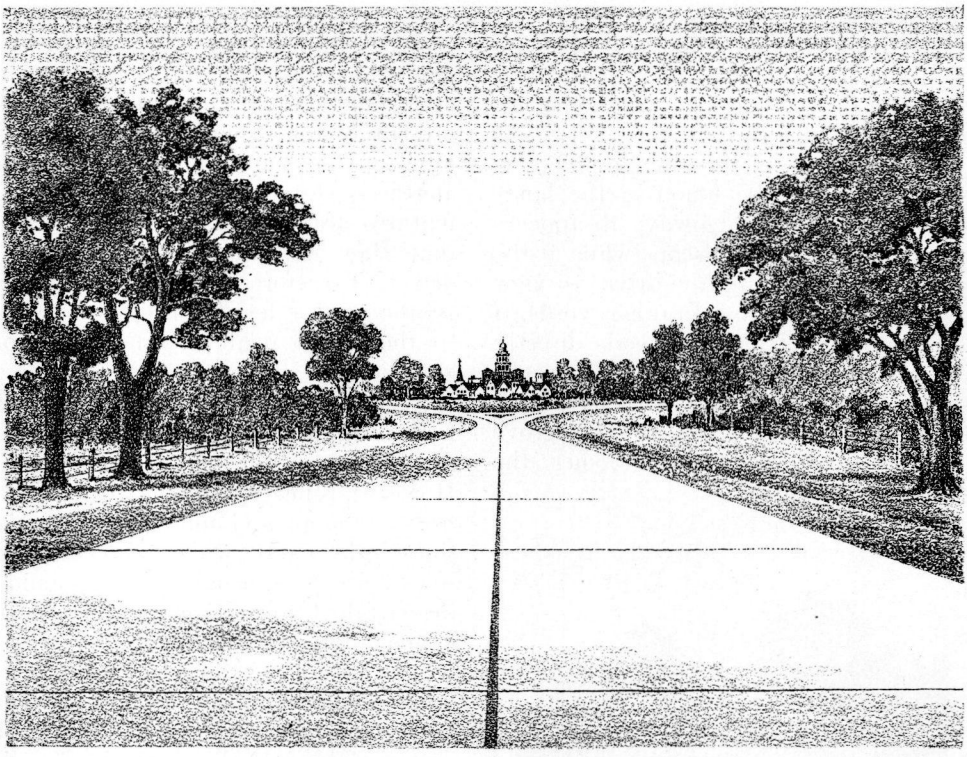


Figure 4. The Interesting View of the City Lies Directly Ahead of the Motorist

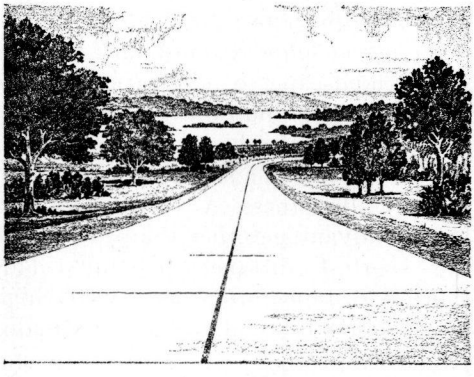


Figure 5. A Direct Approach Overlooking a Bluff Toward the Lake Gives an Attractive Vista for the Motorist.

we may with some confidence make use of it in highway location in coordination with roadside development.

It is unquestionably true that roadside visibility is reduced as the speed is increased. To put it bluntly, it may be said that the faster the car goes, the less the driver sees. Each person will undoubtedly have his own normal road focus at 40, 50 or 70 miles, but very few will have the same reaction. Roadside development should be attained not only through planting, but by means of location so that the highway disappears in the landscape, and thereby becomes a part of it.

REPORT OF COMMITTEE ON LIGHT AS AFFECTING HIGHWAY TRAVEL AT NIGHT

BURTON W. MARSH, *Chairman*

Director, Safety and Traffic Engineering Department, American Automobile Association

SYNOPSIS

In April, 1939, a comprehensive statement on highway lighting was prepared following appointment by the Highway Research Board of a working committee and selection of a number of studies which should be made.

Recent studies of driver behavior both in daytime and after dark by the Public Roads Administration are adding data to that already available relating to light as it affects night highway travel.

A day-and-night study on a one-mile lighted section of U. S. Route 422 near Chagrin Falls, Ohio, reveals some interesting data. They are based on partially completed analyses, and certain conditions at the location studied not considered normal must be taken into account in interpreting their significance. Further studies on longer illuminated stretches and under different conditions may prove they are not typical.

1. Average night speeds were somewhat lower than average day speeds, both without and WITH highway illumination.
2. The percentages of cars traveling faster than 45.5 m.p.h. were smaller at night than in the day, both without and with highway illumination.
3. At night, the average speed was slightly less with highway illumination than with no fixed illumination.
4. At night, with fixed illumination, a wet pavement did not result in a reduced average speed, though the percentage of cars exceeding 45.5 m.p.m. was lower for the wet pavement.

Night traffic fatalities decreased about one-fifth from 1937 to 1938. The decrease for daytime deaths was somewhat smaller.

Nevertheless, nearly two out of three traffic fatalities still occur at night despite the much lower mileage driven after dark. Based on mileage, the motor vehicle death rate is three times as high at night as in the day.

Night accidents as a class are more serious than daytime crashes.

The need for doing something to reduce the night toll is therefore apparent. While there are a number of factors which help increase the night toll, visibility is unquestionably the primary factor. If a prudent driver in normal conditions can see well, he can protect himself against most hazards. Hence major aspects of the problem are light and driver behavior.

HIGHWAY LIGHTING RESEARCH

Last year, following a conference of 31 specialists, the Highway Research Board decided to explore these problems of driver behavior at night and of light as it affects night highway travel.

A working committee was appointed which outlined a series of studies which should be made. These were reported at last year's meeting and will not be repeated now, though the above review was deemed advisable for purposes of orientation. In April, 1939, a quite comprehensive general statement concerning the project was prepared, and is available to those interested. The purpose of this paper is to report progress made.

RECENT STUDIES

While studies have been made in a number of locations, major known prog-

ress has resulted from the work of two agencies—the Public Roads Administration and the State of New Jersey.

Extensive studies of driver behavior both in daytime and after dark, are being made by the Public Roads Administration. Ingenious devices have been developed to make analyses of volumes, speeds, time spacing between cars, transverse positions of cars, overtaking and passing practices, etc. Most of the studies to date have been on unlighted two-lane rural highways without roadside reflectors.

These studies will provide very valuable information for this research by indicating what normal practices are without roadside reflectors or fixed illumination aids. And it should be emphasized that for all except a very small but important percentage of road mileage, drivers will have to continue to depend on headlighting for night seeing.

Hence there should also be reported the highly significant development of the new sealed-beam headlamp. This new design was developed as a joint undertaking of automobile and headlamp manufacturers and of the American Association of Motor Vehicle Administrators. Nearly all of the 1940 models have the new head-lighting, which is generally regarded to be a substantial improvement over any of the earlier headlights.

CHAGRIN FALLS, OHIO, STUDY

The Public Roads Administration has recently cooperated further in a similar day and night study on the one-mile lighted section of U. S. route 422 near Chagrin Falls, Ohio. In this study, the General Electric Company also cooperated, both in extending the period of illumination of this demonstration installation of their equipment and in arranging for the lights to be turned off certain nights so that studies could be made for both lighted and unlighted conditions.

The main purpose of the study was to determine to what extent driver behavior varied under conditions of daylight and night with and without lights. To these variables there was unexpectedly added a variety of weather conditions, including rain and snow.

Excerpted from a progress report of the Public Roads Administration is the following description of the location, the study equipment used and some of the kinds of findings which will result from complete analysis:

"The highway, as it approaches the lighted section, is a horizontal tangent, slightly undulating. About a fourth of a mile from the first light and within the lighted section, the road rises slightly and then drops sharply on about seven per cent grade for a distance of approximately one-half mile. There are two horizontal curves on this grade, one of them being very sharp. The remaining one-fourth mile of the section is approximately a level tangent.

"The pavement is portland cement concrete in fairly good condition, having a nominal width of 20 feet increasing to 27 feet on the hill. The shoulders on the 20-foot pavement are 10 feet wide, flat and well seeded. A two-foot width adjoining the pavement is graveled clay. The 27-foot width on the hill has a six-inch curb.

"The lighting consists of incandescent lamps in specially designed reflectors mounted 25 feet high, 125 feet apart, and extending five feet out over the edge of the pavement. All lights are on the same side of the road.

"Three types of traffic study equipment were used: The passing study equipment, the speed-placement equipment, and the newly-developed speed counter. Some duplication of information will result from this practice which will serve as an accurate check on the equipment and methods employed.

"The speed counter was operated continuously, resulting in a complete speed record by hours of all vehicles, classifying them into twenty speed groups. This machine in its present form is capable of recording the speed of over 95 per cent of the traffic and the results are highly valuable in showing the distribution of speeds. A summary of the results of this portion of the study is included as a part of this report.

"The passing study equipment was set up on the tangent at the top of the hill, beginning at

a point 125 feet ahead of the first light and extending over the rise and down to the point of curvature of the first curve, which was also the point of widening of the pavement. Only two of the recording units were used because of topographic conditions. Consequently, the passing study section covered about one-third mile of highway on a portion of which passing was restricted by limited sight distance. Because of this short distance, and the low volume of traffic prevailing, the number of passings was naturally quite low. The equipment was operated during the afternoons and evenings until about 10 o'clock for four days. The lights were off on alternate evenings. A count of the passings for seven hours of operation on one afternoon and evening reveals that only 37 passings were recorded. The total number, therefore, will probably not exceed 150 and the analysis will not require any great length of time.

"However, it is believed that other information available on these charts will be of considerably more importance, or at least of greater interest than the passings themselves. It will be possible to note the changes in speeds as cars enter the lighted section and the variations in speeds and spacings between vehicles anywhere in the section. The observance of a "Hill" sign near the end of the study section, as exhibited by the number of cars and trucks slowing down at that point, the effect of a 35 m.p.h. speed limit sign on the opposite side of the road, and the indifference to the double-striped no-passing zone will make interesting studies in themselves.

"The speed-placement study was operated at three locations. Two days were spent with the detectors about midway of the passing study sections, this being the location at which the speed counter was operated. The third and fourth days were spent at the center of the sharp curve on the grade, and on the fifth day, the equipment was operated on the level section at the bottom of the hill. The hours of operation were about the same as those for the passing study. These studies will show, not only the effect of illumination on vehicle speeds, but also how the transverse position varies with the ability of the drivers to see the pavement edge."

The said progress report warns of certain conditions which must be taken into account in interpreting the significance of results, as follows:

"First, the traffic volumes are relatively low, and the effect of lights on the speed of traffic as shown here may be quite different from that under heavier traffic conditions that normally might be expected on sections of road justifying illumination (from a volume viewpoint). Second, a variety of topographic conditions in the short length of road probably prevented drivers from assuming a "normal" behavior under any one condition. Third, the novelty of the lights being off for the first time in several years may well have had an influence. Fourth, the number of vehicles for which the speed was recorded at night is not great, although probably these records exceed in number and accuracy any heretofore collected. Fifth, the final criterion of effectiveness of such an installation should be the effect on safety and 'before and after' accident records. Finally, a 35 m.p.h. speed limit was marked, though not enforced on the section."

The speed data have been most completely analyzed at this time. Bearing in mind the cautions above listed, there are indicated several interesting points, a number of which are summarized in Table 1.

TABLE 1

SPEED STUDY DATA

One Mile Lighted Section of U. S. Route 422, Near Chagrin Falls, Ohio

	Average M.P.H.		Percentage Over 45.5 M.P.H.	
	Road dry	Road wet	Road dry	Road wet
Day	38.5	36.5	20.3	12.1
Night, highway lighting on	35.0	35.8	12.4	8.4
Night, highway lighting off	36.8	....	13.0	....

The following points shown in Table 1 warrant attention:

1. Average night speeds were somewhat lower than average day speeds, both without AND WITH highway illumination. Apparently good visibility did not produce a general temptation to speed up.
2. The percentages of cars traveling faster than 45.5 m.p.h. were smaller at night than in the day, both without and with highway illumination. The tendency to drive at high speed was not increased by good lighting.
3. At night, the average speed was slightly less with highway illumination than with no fixed illumination, though the difference was not significant.
4. At night with fixed illumination, a wet pavement did not result in a reduced average speed, though the percentage of cars exceeding 45.5 m.p.h. was lower for the wet pavement.

From 6 to 8 P. M., the night period of heavier traffic, there was a somewhat higher speed on the dry pavement. From that time on throughout the night, the detailed tables showed that there was a somewhat higher speed on the wet pavement. This apparently illogical higher speed as traffic thins out may be due to the fact that when the pavement is wet the gain in visibility upon entering the lighted section is relatively much greater than when the pavement is dry.

Further studies in longer illuminated stretches and under different conditions may prove that these interesting indications are not typical.

Unfortunately, no satisfactory before-and-after accident data are available for this section of highway nor for stretches of highway just beyond the lighted section. It is known that no fatality or serious accident has been reported involving a car leaving this lighted section.

FURTHER STUDIES PLANNED

Further progress on this lighting research project is now assured. The Public Roads Administration is starting a study of existing information relating to the project. In addition, it appears likely that a research specialist will be assigned to prepare quite specific plans for studies which are needed.

DEPARTMENT OF MATERIALS AND CONSTRUCTION

C. H. SCHOLER, *Chairman*

REPORT OF COMMITTEE ON FILLERS AND CUSHION COURSES FOR BRICK AND BLOCK PAVEMENTS

J. S. CRANDELL, *Chairman*

Professor of Highway Engineering, University of Illinois

BRICK FILLER SURVEY IN OHIO

SYNOPSIS

This report summarizes a study of the changes which have taken place in the physical characteristics and conditions of the five petroleum asphalt fillers in the Brick Filler Test Road in Hocking County, Ohio, during the first four years of service.

The results indicate that after four years of service all the fillers showed an appreciable increase in softening point, decrease in ductility and penetration and increase in asphaltene content; that exuding increased with decrease of softening point and increase of 46° C. penetration.

Among the fillers used in this test road bond of filler to brick and low-exuding qualities seem incompatible.

The construction and results of tests on this test road have been previously reported to the Highway Research Board.[1] On June 28, 1939 a complete survey was made on the asphalt filled portions of this pavement. Observations were made on the pavement surface condition; also small areas of the bricks were removed, the adhesion of filler to brick was noted and samples of filler secured. The fillers were prepared for testing by cleaning and melting at the lowest possible temperature to remove water and were then decanted and strained to remove other foreign matter. The fillers were then tested according to standard procedure for brick joint fillers.

In Tables 1, 2 and 3 are shown the results of tests on the "Recovered" filler samples compared with the results of tests on the "Original" fillers as used in the construction of the pavement in 1935.

In Table 4 certain original properties of the fillers in 1935 are tabulated in relation to the behavior of the fillers after a four year service period.

[1] *Proceedings*, Highway Research Board, Vol. 15, p. 174; Vol. 16, p. 182; Vol. 17, p. 349; Vol. 18, p. 221.

RESULTS

1. After four years of service all fillers showed an appreciable increase in softening point, decrease in ductility and penetration and increase in asphaltene content. However, the magnitude of the changes in these factors did not show any consistant relationship with the type of the petroleum base.

2. Within the limits of this study, exuding increased with decrease of softening point and increase of 46°C penetration. Filler No. 6 is a satisfactory low-exuding filler.

3. Among the fillers used in this test road, bond of filler to brick and low-exuding qualities seem incompatible. Bond of filler to brick was found only in the case of Filler No. 9, the most heavily exuded filler, and even in this case the bond was not of sufficient extent to produce water-tightness. From observations on the pavement, it seems that low-exuding characteristics are more important than strong bonding characteristics. No loose brick were observed in any section which indicates that a bituminous filler may perform satisfactorily without bonding to the brick.

TABLE 1

Filler No.	Petroleum base	Softening point (R & B)			Ductility at 25°C		
		Original	Recovered	Increase	Original	Recovered	Decrease
		Deg. C	*Deg. C*	*Deg. C*			*Percent*
6	65 percent Midcontinent / 35 percent Asphaltic	102.0	111.2	9.2	2.25	1.8	20.0
11	Asphaltic	86.0	99.9	13.9	3.0	2.7	10.0
10	Midcontinent	87.0	105.1	18.1	3.6	2.3	36.1
8	Midcontinent	78 4	92.3	13.9	4.3	3.5	18.6
9	Asphaltic	73 6	86.4	12.8	5.0	3.6	28 0

DISCUSSION

Softening Point: All fillers increased in softening point during the four years of service, Filler No. 6 being least effected and Filler No. 10 being most effected. The average increase in softening point for the five fillers was 13.6°C.

Ductility: All fillers decreased in ductility during the four years of service, Filler No. 11 being least effected and Filler No. 10 being most effected. The average decrease in ductility for the five fillers was 22.5 percent.

TABLE 2

Filler No.	Petroleum base	Penetration								
		0°C			25°C			46°C		
		Original	Recovered	Decrease	Original	Recovered	Decrease	Original	Recovered	Decrease
				Percent			*Percent*			*Percent*
6	65 percent Midcontinent / 35 percent Asphaltic	18	13	27.8	24	20	16.7	37	30	18 9
11	Asphaltic	21	16	23.8	31	23	25.8	51	39	23.5
10	Midcontinent	17	12	29.4	26	20	23.1	48	34	29.2
8	Midcontinent	17	14	17.6	30	26	13.3	59	48	18.6
9	Asphaltic	19	13	31.6	31	27	12.9	62	51	17.7

DISCUSSION

All fillers decreased in penetration during the four years of service. Filler No. 8 seems to have been least effected and Filler No. 10 seems to have been most effected. In general, the penetrations of the asphalts decreased about 22 percent.

TABLE 3

Filler No.	Petroleum base	Percent bitumen insoluble in 86°B. naphtha		
		Original	Recovered	Increase
				Percent
6	65 percent Midcontinent / 35 percent Asphaltic	31.6	40.3	27.6
11	Asphaltic	33.0	36.7	11.2
10	Midcontinent	29.0	38.8	33.8
8	Midcontinent	28.9	36.6	26.7
9	Asphaltic	28.5	37.0	29.8

DISCUSSION

From these data it was concluded that the asphaltene content of all fillers increased during the four years of service. Filler No. 11 showed the smallest increase and Filler No. 10 showed the largest increase. The average increase in asphaltene content for the five fillers was 25.8 percent.

TABLE 4

FILLER PROPERTIES VS. BEHAVIOR IN PAVEMENT

Filler No.	Petroleum base	Softening point	Ductility at 25°C	Penetration at 46°C	Behavior in pavement
		Deg. C			
6	65 percent Midcontinent / 35 percent Asphaltic	102.0	2.25	37	Light exuding (Figure 1). No bond to brick.
11	Asphaltic	86.0	3.0	51	Medium-Light exuding (Figure 2). No bond to brick (Figure 3).
10	Midcontinent	87 0	3.6	48	Medium exuding (Figure 4). No bond to brick.
8	Midcontinent	78 4	4 3	59	Medium-Heavy exuding (Figure 5). No bond to brick.
9	Asphaltic	73.6	5.0	62	Heavy exuding (Figure 6). Strong bond to 75 percent of brick side (Figure 7).

DISCUSSION

This table compares certain properties of the fillers as determined in the laboratory at the time of the pavement construction with the appearance of the fillers after four years of service. The fillers are ranked above according to the degree of exuding. It is seen that the degree of exuding varies directly as the 46°C penetration and varies inversely as the softening point. In the above, the petroleum base source does not bear a relation to the degree of exuding. Bond of filler to brick was found only in the case of Filler No. 9 and even in the case of this filler the bond was not 100 percent.

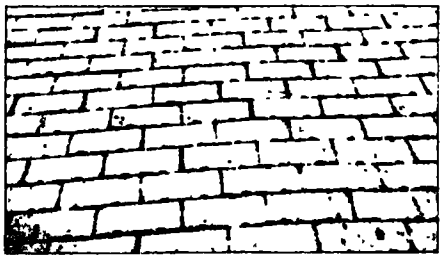


Figure 1. Light Exuding, No Bond, 65 percent Midcontinent—35 percent Asphaltic



Figure 2. Medium-Light Exuding, Asphaltic

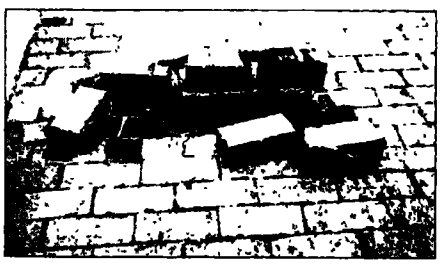


Figure 3. No Bond to Brick, Asphaltic

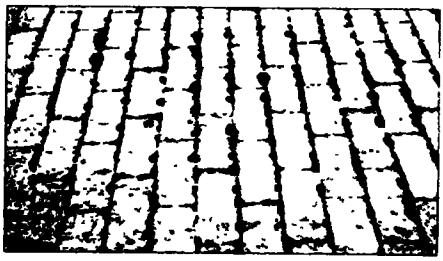


Figure 4. Medium Exuding, No Bond, Midcontinent



Figure 5. Medium-Heavy Exuding, No Bond, Midcontinent



Figure 6. Heavy Exuding, Asphaltic



Figure 7. Strong Bond to 75 percent of Brick Side, Asphaltic

DURABILITY INVESTIGATIONS

By A. T. Goldbeck

Engineering Director, National Crushed Stone Association

SYNOPSIS

The purpose of this paper is to show the need for a rather comprehensive investigation of the phenomena relating to durability of concrete. At the present time, freezing and thawing tests are made in different laboratories, but the methods differ widely and they give different results. Similarly, aggregates are tested with various forms of artificial accelerated soundness tests. It is believed that certain factors regarding aggregates and regarding conditions of exposure are being overlooked in connection with laboratory investigations of soundness. Attention is called to some of these phenomena with the purpose of stimulating discussion which may lead to a clarification of the needs in connection with soundness testing.

For years we have been testing the durability of aggregates by the use of sodium sulfate and more recently with magnesium sulfate with the idea that the action of these salts in crystallizing within the pores of the aggregate will simulate the action of ice. We apparently haven't been any too sure of just how these tests should be made, for we have changed the methods several times and suggestions for still further changes have been made within the past year. Further—the correlation between these tests and service records doesn't seem to be any too consistent and in fact at times they do not correspond at all.

Similarly, we have been struggling with freezing and thawing tests over a long term of years and we still find widely different ideas as to how an accelerated freezing test should be performed. We use different rates of freezing and of thawing, different sizes of specimens, widely different kinds of apparatus and we don't seem to pay much attention to the kind of freezing the structure will get in service. After we get the test results we wonder what they mean in terms of the prospective performance of the aggregate or concrete for the particular conditions for which it is intended.

It is an easy matter to be arbitrary and simply decide how these tests should be made and then decree what test limits represent failure. That, in large degree, is what we have done in the past and we shall be forced to continue our arbitrary methods until we obtain some really fundamental and comprehensive information upon which to base our tests and specification limits.

But learning the proper method for performing accelerated soundness tests is only a small part of the information needed in connection with the durability of aggregates and of concrete. We should undertake a basic study to find out what are the various influences affecting durability.

When concrete fails upon exposure to the weather, it has been subjected to internal stresses greater than those it can withstand, or its constituents may have suffered chemical change. While it is true that the expanding action of ice within the pores of concrete or of aggregate may exert a disrupting effect, this expansion phenomenon does not tell the full story of the cause of disruption. Let us ask ourselves some questions which seem to bear on this subject.

1. Freezing and thawing are accompanied by temperature change in the concrete. How important may temperature change alone be in causing failure?

   (a) By virtue of different coefficients of expansion of constituent materials.

(b) Because of differential stresses between the surface and the interior of the concrete.

2. May not rapidity of change in temperature be highly important?

3. What about the effect of type of water exposure? Concretes have different exposures such as continuous intermittent wetting and drying, exposure on one side to moisture and the other side to freezing.

4. How important is the thermal expansion of ice within the pores of concrete after it has once formed? Ice, while increasing in temperature up to the melting point expands about four times as much as concrete. Its coefficient of linear expansion is 0.000028 as against only 0.000006 for concrete. Is this important and under what circumstances?

5. Sudden changes in temperature can cause high stresses and cracking of concrete surfaces. How significant is this phenomenon in producing failure?

6. Should we consider other characteristics of aggregates as well as freezing resistance in connection with durability studies? For instance, how is the bond between mortar and aggregate affected by smoothness of surface, by thermal expansion and by shrinkage and swelling of the mortar under changing moisture conditions?

7. Some aggregates have compounds which expand upon long exposure to moisture. Surely something different than sodium sulfate or freezing and thawing is needed to detect that kind of unsoundness.

These are only suggestions as to a very few of the phenomena which might influence failure. There are many more. They indicate the probability that some failures attributed to freezing and thawing may be due to other factors. They suggest that a single standard test for making an accelerated weathering test of aggregates may be woefully lacking and this applies to concrete also. They also show that the degree and kind of exposure need taking into account in considering the degree of durability necessary in concrete or in aggregates.

It would seem desirable that we begin to study durability in its fundamental aspects so that we might be able to write accelerated test methods which will be significant and which we might use with confidence in our investigations and in our specifications. Such a study will not be simple. It will require many separate studies of many different kinds. Steps leading to this study are already being considered by the American Society for Testing Materials and in this effort the Highway Research Board should play its part also. A large cooperative study of the basic causes of lack of durability is surely needed and it is urged that the Department of Materials organize investigations in this field.

SOME OBSERVATIONS REGARDING FROST ACTION

By A. F. Gill and J. F. J. Thomas
National Research Council of Canada

SYNOPSIS

The authors advance the interesting hypothesis that the nature of frost heave in roads, the spalling of concrete pavement, and the disruption of masonry through frost action is substantially the same. It has been established that frost boils and the greater part of frost heaving is a result not so much of freezing of moisture present in any one spot in soil capillaries but of the building up of ice plates through migration of moisture from road shoulders and sub-grade water tables.

The spalling of concrete and masonry may be due not to the freezing of static water within the pores of the mass, but to the building up of ice plates or incipient ice plates due to migration from the warm side. To investigate such a conclusion, a test was devised that duplicated as nearly as possible the natural conditions. As a first step, standard building bricks were tested with one end in water above the freezing point, the other end exposed to a temperature between 0 and 10 deg. F., and the mid-section of the brick surrounded by insulation.

Preliminary tests on a porous brick seem to bear out the assumptions, for the brick spalled quickly near the cold end, producing a clean fracture which was coated with about 1 mm. of continuous ice.

The action of frost on soil masses has long been recognized as a major cause of deterioration of highway surfaces and it has been studied by numerous laboratories and field investigators, particularly during recent years. At the 1938 meeting of the Highway Research Board a very informative session on prevention of frost heave was held and the consolidation of the papers then presented, as published in the proceedings, supplies very interesting reading to anyone with more than a passing interest in the subject. In addition, laboratory investigations, reported from time to time, have served to establish certain principles which may now, it is believed, be taken as fundamental.

Detrimental frost action manifests itself in two main ways. First there is the frost heave in which vertical and, to a lesser extent, horizontal movement of soil-surfaces takes place during the process of freezing. Secondly, there is what is commonly known as the frost boil which often does not appear until after thawing of the road surface has set in. It is characterized by the development either of an isolated eruption or of a comparatively widespread morass. Both of these phenomena are caused by the action of frost in the presence of moisture. Under ordinary conditions mere freezing of sub-grade moisture in situ is not a significant cause of either frost heaves or frost boils. In order that the effect may be appreciable, it is necessary that conditions be such as will allow the migration of moisture from sub-grades or adjacent areas such as road shoulders to build up ice plates at the freezing surface. The migration of water in this way is a recognized physical phenomenon and, other things being equal, it is a function of the capillarity of the medium through which the water must move.

The resultant ice plates may be of a lamellar nature, or they may be the well-known "Jack Frost" needles. In either event greater or less surface movement results. If weather conditions are such that thawing of the frozen soil takes place from the top downward—and this need not always occur—the time comes when there is a frozen and relatively impermeable sub-grade, surmounted by a soil mass containing the excess water

resulting from the melting of the ice plates. Thus, is brought about the frost boil.

The conditions necessary to detrimental frost action on soil surfaces are thus, in brief, a supply of water that is free to migrate and a temperature gradient running from above the freezing point of water to somewhat below it. The recognition of these facts—for which we must thank the students of soil phenomena—has, it is believed, more than passing significance to those interested in concrete road construction and also to those concerned with the action of frost on masonry in buildings and other structures. It is recognized that under some conditions concrete highways have been known to spall, and also that some brick and other building units, particularly those of the porous types—although not necessarily all—have characteristically low resistance to frost action.

Frost is also recognized as one of the major reasons why rock formations, whether igneous or sedimentary, are eventually broken down into earthy material. Most of us have been told at a very early age that the action of frost under such circumstances is due to the fact that ice occupies approximately one-seventh more volume than water. The trouble is that when water-saturated materials freeze they sometimes spall and sometimes do not. The laboratory investigation of building units and of concrete materials generally has been largely along the lines of conventional freezing and thawing tests. It is believed that it is a fair statement to make that the results of such tests must be interpreted with a great deal of caution.

The view that it is mere expansion of water into ice that exerts the disrupting effect on a saturated, porous matrix led in early studies to the belief that a more porous body would be more susceptible to frost action. The fact that this was not always true has been explained in some measure by the conception of a saturation coefficient. In other words, certain stones or masonry units, because of their chemical or physical nature or of the structure of their voids, do not become completely filled with water when apparent saturation has been reached. As a result, when freezing occurs there is still room left for the water to expand and no harm is done. Even with this rather elastic provision the forecasting of the behaviour of stone or masonry on the basis of laboratory tests cannot be considered to be very satisfactory. It is probably fair to say that materials that are relatively very dense do show better frost resistance, but this, it is believed, is only part of the story.

In studies that have been made of the deterioration of masonry structures, particularly by the Department of Scientific and Industrial Research in England, some rather interesting observations have been made. For example, masonry monuments and other structures prior to, say, one hundred years ago, were generally laid up in mortars of comparatively high porosity or, more accurately, probably, permeability. In numerous instances these structures have given very good accounts of themselves from the ageing and weathering point of view. However, in recent years it has been observed that when such structures were repointed with dense mortar, the effect upon the unit so bonded was often disastrous, in that spalling immediately set in. The explanation offered was that the spalling was due to migration of soluble salts derived either from the mortar, from the unit, or possibly through the action of atmospheric sulphur gases on calcareous materials; with a permeable mortar the soluble salts tended to migrate through the mortar and, as they crystallized, the mortar spalled off; when a relatively dense mortar was used migration tended to take place through the

building unit with the result that the spalling, when it occurred, did much more damage.

It is believed that the concentration of soluble salts in quantities sufficient to disrupt the masonry, is closely parallel to the concentration of ice in similar localities brought about by similar migration.

In parts of Canada where the temperature stays well below the freezing point for several months of the year with only short periods of thaw, it is sometimes possible to remove a brick or a section of stucco from the exterior of a building and find a layer of ice in the interior of the wall. Such ice might originate from rain or snow or from moisture coming through from the inside, reaching its dew point, condensing, and eventually becoming solid. In either case there is an accumulation at what is presumably the "frost line."

Consideration of the conditions bearing upon the weathering of soil masses, concrete road surfaces, masonry buildings and masonry dams and retaining walls, shows that they have in common the factors mentioned above as being necessary to detrimental frost action, i.e., a temperature gradient from above the freezing point to below it, a degree of permeability, and moisture available on the warm side to migrate to the point at which freezing is taking place.

On this basis the criterion of the weathering resistance, so far as frost is concerned, of any material, would not be so much a matter of its porosity or absorption, as of its permeability. As is well known, these characteristics need not necessarily parallel one another. The measurement of permeability is not always an easy task, particularly in ceramic or cementitious products. On the other hand, the most reliable criterion of the suitability of any material for a given purpose is the test that most closely reproduces the sequence of conditions that it must withstand in service in their proper relative magnitude. According to the above reasoning the conditions that should be observed in any test of the stability of a road mass, a concrete slab, or a masonry structure to frost action would be those already outlined.

In preliminary tests made in these laboratories nearly a year ago, the indications were that it would be possible to test building units and concrete specimens in accordance with these conditions. A thermostat was constructed in which one or more brick or concrete specimens could be stood on end and partially immersed in water, the upper end being exposed to the air of a cold room at a temperature considerably below the freezing point. The specimen was surrounded with thermal insulation at the point where it left the water, so that no difficulty was encountered in keeping the water above the freezing point. In this apparatus it was found possible to spall more porous types of brick in the course of less than 48 hours and the formation of an ice plate at the freezing point was plainly discernible.

Some brick failed to break in the test but no quantitative data were available as to the relative permeability of the brick nor was there any information on their behavior in service.

EXPERIMENTS IN CURING CONCRETE PAVEMENTS

By Bert Myers

Engineer of Materials and Tests, Iowa State Highway Commission

SYNOPSIS

Thirteen years of experience in curing concrete pavements with a great many types and kinds of cures form the background for this report. Among those methods used were wet earth, wet burlap, various bituminous types, calcium chloride both externally and internally, the Hunt process and others. From these several experiments a few brief conclusions were drawn as follows:

1. Earth and water curing provides the most desirable results of any that were studied.
2. The curing treatment applied during the first few hours after the concrete is placed has the greatest effect upon its strength.
3. The effect of a curing method upon strength may be predicted from laboratory tests for prevention of moisture loss.
4. While under the climatic conditions existing in Iowa, differences of strength due to differences in curing methods tend to decrease with the age of the concrete, they do not disappear entirely for a very long time.
5. The difference in slab length due to differences in methods of curing tend to disappear with increasing age of the pavement.

The term "curing" as applied to concrete is understood to mean its artificial environment during some relatively brief period immediately following its placement. It is intended to provide conditions favorable to the development of desirable characteristics in the material. These favorable conditions are believed to be a reasonably high uniform temperature and the prevention of the rapid loss of moisture. Specifications for curing are influenced by such practical considerations as cost, convenience and time. It is generally conceded that an ideal method for curing concrete pavements would be to cover the surface as quickly as possible with some light material which is kept wet, followed after the concrete has hardened by some material with considerable heat insulating value which is also kept continuously wet for a period of a month or more. However, such a method is not always practical from considerations of cost and convenience. For this reason there has been for several years intensive study of methods of curing that might prove satisfactory alternates for the method conceded to be good.

The Iowa State Highway Commission began extensive studies of the curing of concrete pavements 13 years ago and has continued these studies to the present. Much of the information accumulated has been reported to committees of national technical societies. Within the past few months some interesting observations have been made as to the relative effects of various methods of curing after a lapse of several years. This report contains a brief summary of the earlier work and a more detailed report of the more recent observations.

FIELD TESTS—CALCIUM CHLORIDE ADMIXTURE

The fact that the addition of calcium chloride to concrete hastens its hardening suggests that its use as an admixture might shorten the time during which other curing treatment is necessary.

Of 9.5 miles of pavement built in Mahaska County in the fall of 1926, a part was cured with wet burlap for one day followed by wet earth for an additional nine days. The concrete in the remainder contained 2 lb. of calcium

chloride per bag of cement and was cured with wet burlap for one day. The 17 miles built in the spring and summer of 1927 were cured in the same way except that the concrete containing the calcium chloride admixture was kept covered with wet burlap for two days. Beam specimens 6 by 6 by 30-in. and 6 by 12-in. cylinders were molded as the work progressed and placed on the pavement surface when removed from the molds to receive the same curing treatment as the pavement.

For the materials used and the conditions surrounding this project, it appears that neither method of curing was consistently superior in compressive strength.

Surface Scaling

In order for a method for curing concrete pavements to be satisfactory it must have no unfavorable effect upon the pavement surface. In the winter of 1928-29 a survey was made of 1630 miles of pavement built in the years 1918 to 1928 inclusive, to observe the relative amounts of surface scaling classified according to the curing methods. In this survey a scaled area having a dimension of 1 ft. measured parallel to the center line of the pavement would be reported as 1 ft. of scaled pavement regardless of the dimension of the scaled area in any other direction. Table 2 gives the comparison for pavements built in 1926, 1927 and 1928.

From the figures in Table 2 it appears that certain conditions conducive to scaling must have existed on the work done in 1926 which did not prevail in the two following years. It is remembered that in 1926 whenever the finishing crew experienced difficulty in finishing, they were in the habit of sprinkling water on the surface of the concrete. Difficulty in finishing was encountered more generally with concrete that contained calcium

TABLE 1

EFFECT OF METHOD OF CURING ON COMPRESSIVE STRENGTH—MAHASKA COUNTY PROJECT

Year built	Aggregate	Compressive strength—lb. per sq. in.			
		Cylinders		Cores	
		Earth cured	$CaCl_2$ admixture	Earth cured	$CaCl_2$ admixture
1926	Limestone	(23) 4290	(54) 4667	(10) 4390	(39) 4452
1927	Limestone	(58) 5609	(57) 5073	(23) 6503	(23) 6411
1927	Gravel	(25) 5274	(47) 5280	(11) 6459	(26) 5737

NOTE: The figures in parentheses indicate the number of specimens tested.

STRENGTH TESTS

The uncontrolled variables, such as weather and variations in construction procedures which are likely to be encountered in any field study of this kind seem to have influenced the results of this study, particularly the transverse strength tests. In this case it appears that although at one day the calcium chloride transverse strengths were greater, at the age of 90 days for the concrete placed in the fall of 1926 and at the age of 21 days for that placed in the spring and summer of 1927 the transverse strengths were practically the same for the two kinds of concrete.

The compressive strengths of the field cylinders and of cores drilled from the pavement were more consistent. The average strengths of cylinders tested at 90 days and of cores tested at ages from 12 to 17 months are given in Table 1.

chloride than with plain concrete. Strict instructions against this sprinkling of the surface were issued early in 1927.

Table 3 gives the results for all the methods of curing used in the ten-year period.

TABLE 2

RELATIVE EFFECT OF CALCIUM CHLORIDE ADMIXTURE AND EARTH CURING UPON SCALING

Year built	Method of curing			
	Wet burlap plus wet earth		$CaCl_2$ integrally plus wet burlap	
	Miles laid	Percent of length scaled	Miles laid	Percent of length scaled
1926.....	65.7	4 25	24.1	27.87
1927.....	205.9	0..67	100.8	0.64
1928.....	623.8	0.18	77.6	0.03

gravel used in this concrete was composed almost entirely of siliceous particles.

TABLE 3

EFFECT OF METHOD OF CURING ON SURFACE SCALING

Pavements constructed 1918 to 1928 inclusive, surveyed in winter of 1928-29

Method of curing	Miles surveyed	Percentage of total length showing scale
Wet canvas or wet burlap plus wet earth.....	1406	1 35
Calcium chloride integrally plus wet burlap..... ..	203	3.65
Wet canvas or wet burlap plus calcium chloride on surface.....	18	15.06

TABLE 4

RELATIVE EFFECT OF CALCIUM CHLORIDE ADMIXTURE AND EARTH CURING ON AVERAGE SLAB LENGTH

Year built	Coarse aggregate	Average slab length—ft.			
		1929 survey		1939 survey	
		Earth and water cured	$CaCl_2$ and wet burlap cured	Earth and water cured	$CaCl_2$ and wet burlap cured
1926	Limestone.....	50.3	22.1	....	....
1926	Gravel.....	....	18.1	....	....
1927	Limestone.....	71.9	21.9	36.7	17.6
1927	Gravel.....	39.7	19.5	25.3	16 9

Table 4 gives the results of observations of the spacing of transverse cracks on the experimental project at two times approximately ten years apart. The pavement was built without transverse joints.

The 1929 survey covered the entire project. The 1939 survey covered only about two miles of representative sections for each set of conditions. These results indicate that differences in the method of curing caused a difference in the number of transverse cracks which has persisted for 10 years, although this difference is decreasing with age. The

LABORATORY TESTS—CALCIUM CHLORIDE ADMIXTURE

In 1927 some laboratory work was done to determine the effect upon strength of the addition of calcium chloride to mortar and concrete at a rate of 2 lb. per bag of cement. Specimens which contained calcium chloride were cured in the same way as similar specimens which contained no calcium chloride. Seven different brands of cement were used.

The mortar specimens, made with natural sand, was molded into standard

briquettes which were stored in a moist closet for 1 day and then in water until tested in tension at ages of 1, 2, 4, 7, 14, 21, 90 and 180 days. The addition of calcium chloride caused an increase in strength at one day for six of the seven brands of cement. The increase in strength at 180 days due to the addition of calcium chloride was very slight for any brand, and in one case it caused a decrease.

The concrete specimens were 6 by 6 by 30-in. beams which were stored under wet burlap until removed from the molds, after which they were stored in the laboratory air until tested.

The same cements and the same sand as was used in the briquettes were mixed with gravel and with crushed limestone to produce concrete, having the proportions 1:1.93:2.91 by weight with a water-cement ratio of 0.417. Similar specimens were made with and without an admixture of 2 lb. of calcium chloride per bag of cement. They were tested as beams after which the broken ends were squared with a saw to provide 6 by 6 by 10-in. prisms for compressive tests.

The results of compressive tests indicate that with two exceptions, the addition of calcium chloride caused some increase in strength for all cements with both kinds of coarse aggregate at all the ages of test; namely 4, 7, 14, 21, 28, 90 and 180 days. One brand of cement with gravel coarse aggregate showed practically the same strength for the two kinds of concrete at an age of 7 days. Another brand showed a slightly higher strength for plain concrete with limestone coarse aggregate at 90 days.

The fact that the beams were not soaked before testing to produce a uniform distribution of moisture throughout the cross section of the specimen causes some doubt as to the reliability of the transverse tests.

They indicate, however, that adding calcium chloride admixture has a more consistent effect upon compressive than upon transverse strength of specimens subjected to the curing treatment that these received.

A further study of the effect of the admixture of varying percentages of calcium chloride with seven different brands of cement in mortars of three consistencies was made in 1930. Natural local sand was used in the proportions of 1-2.06 by weight. The amount of mixing water used was basic, 1.2 basic and 1.4 basic as described by Talbot and Richart. The amounts of calcium used in the mixture corresponded to 0, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 7.0 and 10 lb. per bag of cement for 5 of the cements while the maximum used with one of the brands was 3.0 and the maximum for the seventh was 4.0. The specimens which were 2 by 2 by 14-in. beams were stored in a moist closet in the molds for 24 hours, then removed from the molds and stored in moist sand for 48 hours, after which they were stored in the laboratory air until 12 hours before testing. During the last 12 hours they were stored in water. Transverse tests were made at 4, 7, 14, 28, 90 and 180 days and compression tests were made on prisms sawed from beam ends at 28, 90 and 180 days.

In these tests the addition of some quantity of calcium chloride caused an increase in strength at some age and at some consistency for every cement. However, the optimum quantity of calcium chloride for one brand of cement at one age and one consistency might cause a decrease in strength for another age or consistency for the same cement. The optimum quantity of calcium chloride admixture varied from ½ lb. to 10 lb. per bag for different brands of cement, age at test and consistency.

The conclusion drawn from this series of tests was that the determination of the

optimum quantity of calcium chloride admixture was so complicated that its use was impractical for the conditions affecting highway work in Iowa.

VARIOUS METHODS OF CURING—1928

In 1928 a more extensive study of curing was made through the construction of 42 concrete slabs each 10 ft. wide, 11 ft. long and 6 in. thick. They were built out-of-doors on the ground near the laboratory in 4 series as given in Table 5.

In order to insure uniform temperature and humidity during the placing and finishing all of the concrete was mixed between 10:00 a. m. and 2:30 p. m.

TABLE 5

Series	No. of slabs	Dates built	Average mean daily temperature	Average relative humidity
I	14	July 31 to August 10.....	88	68
II	12	August 14 to 17.....	87	65
III	11	September 18 to 21.....	66	56
IV	5	November 13 and 14.....	54	..

Of the 15 different methods of curing used, the following are of interest:

1. Wet burlap 24 hr., moist earth 9 days.
2. Wet burlap 24 hr.
3. Wet burlap 72 hr.
4. Wet burlap 24 hr., moist earth 27 days.
5. Wet burlap 24 hr., surface application of calcium chloride at 2½ lb. per sq. yd.
6. Calcium chloride integrally at 2 lb. per bag of cement, wet burlap 48 hr.
7. Wet burlap until initial set, sodium silicate.
8. "Curcrete."
9. Wet burlap 24 hr., sodium silicate.
10. No curing.
11. "Hunt Process."

These slabs were sawed into 6 by 6 by 30-in. beams which were tested in cross bending at 7, 28, 90 and 360 days. Compression tests of 6 by 6 by 10-in. prisms sawed from the ends of broken beams were made at 8, 29 and 90 days.

In 1929 six slabs each 2 ft. wide by 6 in. thick by 80 ft. long composed of the same kind of concrete as used in the 1928 series were constructed out-of-doors near the laboratory for the purpose of studying the effect of method of curing on change in volume. In the summer of 1939 pieces from the ends of these slabs were sawed into 6 by 6 by 24-in. beams and tested for transverse strength. The ends of these beams were squared with a saw to provide 6 by 6 by 10-in. prisms which were tested in compression. Tables 6 and 7 show the relative strengths for the various methods of curing. The values for ages 7 days to 360 days are the averages for the first 3 series in the 1928 work.

The values in Table 6 for ages 7 to 360 days inclusive are each computed from the average of 80 to 89 tests. The values in Table 7 for ages 7 to 360 days inclusive are each computed from the average of 28 to 30 tests. The values for age 10 years in both tables are each computed from the average for 5 tests.

These results indicate that under the climatic conditions to which this concrete was exposed, differences in strength due to the effects of curing methods tend to disappear but that great differences will not disappear entirely for a very long time.

The work described for concrete was supplemented by laboratory tests on

TABLE 6

RELATIVE EFFECT OF CURING METHOD ON TRANSVERSE STRENGTH

Method of curing	Modulus of rupture—lb. per sq. in.				
	7 day	28 day	90 day	360 day	10 yr.
"Standard"—Wet burlap 24 hr. plus wet earth 9 day...	621	748	771	864	967
	Modulus of rupture—percent of "standard"				
Wet burlap 24 hr.	86	82	88	89	...
Wet burlap 72 hr.	92	91	93	92	...
Wet burlap 24 hr. plus wet earth 27 day	95	100	104	101	...
Calcium chloride integrally plus wet burlap 48 hr.	82	84	87	97	...
Wet burlap until initial set plus sodium silicate	82	84	87	92	...
No curing	77	78	83	86	94
"Curcrete"	86	97	101	105	98
"Hunt Process"	90	93	92	96	97
Wet burlap 24 hr. plus $CaCl_2$ @ 2½ lb. per sq. yd.	95	94	95	94	98
Wet burlap 24 hr. plus sodium silicate	86	89	90	94	98

TABLE 7

RELATIVE EFFECT OF CURING METHOD ON COMPRESSIVE STRENGTH

Method of curing	Compressive strength—lb. per sq. in.			
	8 day	29 day	90 day	10 yr.
"Standard"—Wet burlap 24 hr. plus wet earth 9 day	3268	4372	4943	7530
	Compressive strength—percent of "standard"			
Wet burlap 24 hr.	86	80	84	....
Wet burlap 72 hr.	93	91	93	....
Wet burlap 24 hr. plus wet earth 27 day	92	93	96	....
Calcium chloride integrally plus wet burlap 48 hr.	97	101	97	....
Wet burlap until initial set plus sodium silicate	76	78	80	....
No curing	75	73	79	96
"Curcrete"	97	99	95	96
"Hunt Process"	92	90	83	100
Wet burlap 24 hr. plus $CaCl_2$ @ 2½ lb. per sq. yd.	97	93	91	90
Wet burlap 24 hr. plus sodium silicate	86	90	85	94

mortar specimens to determine the relationship between moisture loss and strength.

This work which was reported to the Curing Committee of the Highway Research Board led to the following conclusions:

1. So far as its effect upon strength is concerned, the efficiency of a curing method is measured by the degree to which the method prevents the loss of water from the concrete during its early age.
2. Laboratory studies of mortar under controlled temperature and humidity conditions provide a reliable means for evaluating the efficiency of curing agents.

A part of a paving project built in 1931 was cured with earth and water, a part with an application of asphaltic emulsion containing 35 per cent of asphalt, and a part with an emulsion containing 52.5 per cent of asphalt. The emulsion was whitewashed to reduce the temperature effects of the sun. The results are shown in Table 8.

EFFECT OF DARK COLORED COATINGS ON SPACING OF TRANSVERSE CRACKS

A considerable mileage of pavement built in 1929 was cured by the application of an asphaltic emulsion to the fresh concrete. There was evidence that transverse cracks would develop at more frequent intervals in concrete cured by this method than in pavements cured by some other methods. Therefore, on each project where the emulsion was used, a section approximately one mile in length was cured with earth and water.

TABLE 8

COMPARISON OF LABORATORY AND FIELD TESTS OF THE EFFICIENCY OF A CURING AGENT

Laboratory tests				Field tests		
Emulsion, 52 percent asphalt		Emulsion, 34 percent asphalt		Average compressive strength of cores, age 4 mo.—lb. per sq. in.		
Moisture loss, 6 day	Trans. strength @ 7 day	Moisture loss, 6 day	Trans. strength @ 7 day	Wet earth curing	Cured with 52.5 percent emulsion	Cured with 35 percent emulsion
Percent		*Percent*				
5.5	(5) 694	25	(5) 554	(8) 6494	(8) 6552	(6) 5655

Figures in parentheses show numbers of specimens.

Early in 1929 a specification was adopted by the Iowa Highway Commission covering the use of impervious coatings as curing agents. The acceptability of such curing agents was based on the results of moisture loss tests of mortar specimens subjected to a temperature of 90° to 100° F. and a relatively humidity of 30 to 50 per cent.

RELIABILITY OF LABORATORY TESTS

The reliability of laboratory tests for moisture loss of mortar as a measure of the efficiency of a curing agent may be indicated by the following experience:

Table 9 shows the results of five crack surveys that have been made over this work. The slab lengths reported for the pavement cured with emulsion refer only to the sections contiguous to the sections cured with earth and water and of approximately the same length.

These results indicate that while the method of curing had a considerable effect on the number of visible transverse cracks that developed at early ages, the

difference in slab length due to curing method is diminishing with time.

Table 10 shows the strengths of cores drilled from some of the same sections as those reported in Table 9.

strength in this case is probably due to some cause other than the curing.

Table 11 gives information on the length of slab as affected by some methods of curing used during 1932.

TABLE 9

LENGTH OF SLAB AS AFFECTED BY TIME AND METHOD OF CURING

Pavement Built in 1929, Without Joints. Averages for 6 Projects of Each Kind of Coarse Aggregate

Date of survey		Nov. 1929	Nov. 1930	Feb. 1932	Dec. 1933	Nov. 1939
Coarse aggregate	Curing	Average slab length—ft.				
Limestone	Wet earth....	138	92	83	62	32
Limestone	Emulsion.....	77	53	51	48	28
Gravel....	Wet earth....	103	52	43	35	23
Gravel....	Emulsion.....	43	32	29	26	20

Earth curing consisted of wet burlap for 1 day followed by earth kept wet for 6 days.
Emulsion curing consisted of asphaltic emulsion applied to fresh concrete.

TABLE 10

COMPRESSIVE STRENGTH OF CORES DRILLED FROM PAVEMENT, AS AFFECTED BY TIME AND METHOD OF CURING

Pavement Built in 1929

Project	Coarse aggregate	Average compressive strength—lb. per sq. in.					
		Age 9-12 mo.			Age 10 yrs.		
		A earth	B emulsion	B/A percent	A earth	B emulsion	B/A percent
Carroll...	Limestone	(5) 5914	(9) 5592	94.6	(5) 6906	(5) 6712	97.2
Clarke....	Limestone	(4) 4912	(9) 5333	108 6	(5) 6790	(5) 6954	102.4
Dubuque	Limestone	(5) 5844	(9) 5478	93.7	(5) 8718	(5) 7982	91.6
Jackson .	Gravel....	(5) 5376	(11) 5291	98.4	(5) 7644	(5) 7608	99.5
Jones.....	Gravel....	(7) 5823	(8) 5196	89.2	(5) 7678	(5) 7042	91.7
	Average. .	5574	5378	96.5	7547	7260	96.2

Figures in parentheses show the number of specimens.

The results indicate that while the differences in strength apparent at early ages are decreasing with age, they still exist to some degree after 10 years.

There is some question as to the reliability of the comparison for the Clarke County Project. The specimens from the earth cured section were observed to be much more porous than those from the emulsion cured section. The difference in

PAPER AS A CURING AGENT

Mark Morris reported in 1932 on "The Use of Paper for Curing Concrete Pavement."[1] This report showed that at an age of 7 months cores drilled from sections of a Story County project that had been cured with wet burlap for 1

[1] *Proceedings*, Highway Research Board, Vol. 12, p. 339.

day followed by an application of heavy waterproof paper had a strength 93.8 per cent of similar cores drilled from sections cured with wet burlap and wet earth. Cores drilled from the same sections in 1939 showed that at an age of 8 years the paper cured concrete had 94.5 per cent of the strength of concrete cured with earth and water.

WET BURLAP FOLLOWED BY IMPERVIOUS COATING

Experiments as to the effect of method of curing on length changes which were reported to the Highway Research Board in 1931 [2] indicated that the unusual frequency at which transverse cracks occurred in pavements cured with an asphaltic coating applied to the fresh concrete might be due to the great volume changes caused by the unusual heat absorption of the dark colored surface. This suggested two expedients for reducing this effect.

1. To apply some light colored coating to the bituminous coating as soon as practicable after the bituminous coating was applied.
2. To delay the application of the bituminous coating until the concrete had gained some strength to withstand the stresses induced by volume change due to changes in temperature.

Therefore, the specifications which have applied to the use of bituminous curing agents on highway work in Iowa since 1931 have provided that the concrete shall be cured with wet burlap for one day before the bituminous coating is applied, and that a coating of whitewash shall be applied over the bituminous coating.

It was suggested by those interested in the sale of bituminous material for use as a curing agent that perhaps the initial curing with wet burlap was not necessary if the bituminous coated surface were whitewashed or that the benefits derived from the initial wet burlap were not proportional to its cost.

TABLE 11

LENGTH OF SLAB AS AFFECTED BY METHOD OF CURING
Pavement Built in 1932—Crack Survey in November 1939

Coarse aggregate	Number of projects	Spacing of expansion joints	Curing treatment			
			Earth	Burlap-emulsion-whitewash	Emulsion-whitewash	Paper
			Slab length ft.—Nov. 1939			
		Feet				
Gravel.....	9	60	23	22	..	..
Limestone.....	5	80	34	37	..	..
Limestone.....	1	80	..	23	29	..
Gravel.....	1	60	28	27	26	..
Limestone.....	1	80	49	..	..	50

Earth curing consisted of wet burlap for 1 day followed by earth kept wet for 6 days.
Burlap-emulsion-whitewash curing consisted of wet burlap for 1 day followed by asphaltic emulsion which was whitewashed. Paper curing consisted of wet burlap for 1 day followed by paper left in place for 7 days.
It will be noted that after 7 years the method of curing shows little effect upon slab length.

[2] *Proceedings*, Highway Research Board, Vol. 13, Part II.

For this reason it was arranged to experiment with the two methods of using bituminous materials as a curing agent on two paving projects.

Table 12 gives the results observed at 28 days and at 7 years after the concrete was placed.

These data indicate that the initial curing with wet burlap was responsible for an appreciable increase in the strength of the concrete and that this advantage has not disappeared at the end of 7 years.

Since wet burlap followed by wet earth is the commonly accepted standard for curing concrete pavements it is interesting to study the data shown in Table 13.

These values seem to indicate that when the application of an impervious coating is preceded by an initial curing with wet burlap, strengths approximately equal to those for earth and water curing may be attained.

TABLE 12

EFFECT OF TIME OF APPLICATION OF ASPHALTIC EMULSION CURING AGENT UPON COMPRESSIVE STRENGTH OF CORES

Method of curing	Compressive strength—lb. per sq. in.			
	Boone Co. project		Mitchell Co. project	
	Age 28 day	Age 7 yr.	Age 28 day	Age 7 yr.
Emulsion applied to fresh concrete.....	(15) 4982	(5) 7052	(18) 3944	(5) 6588
Emulsion applied after 24 hr. wet burlap...	(20) 5439	(5) 7784	(25) 4597	(5) 7272

Figures in parentheses show number of specimens.

TABLE 13

EFFECT OF METHOD OF CURING ON COMPRESSIVE STRENGTH OF CORES, 1932

Method of curing	Compressive strength of cores—lb. per sq. in. Age 7 yr.				
	Boone	Grundy	Iowa	Dallas	Marion
Wet burlap and earth.....	(5) 7686	(5) 7465	(5) 8288	(5) 7280	(5) 6970
Wet burlap and emulsion	(5) 7784	(5) 6864	(5) 8164	(5) 7150	(5) 7554
Wet burlap and emulsion cured in percentage of earth cured..	101.3	91.9	98.5	98.2	108.4

Figures in parentheses show number of specimens.

FIELD EXPERIMENTS

Various Methods of Curing—1929

A field experiment conducted in 1929 involving a comparison of earth and water, "Curcrete," "Hunt Process," calcium chloride applied on the surface, sodium silicate and wet burlap curing has provided some information as to the relative effects of these methods of curing. The strength values are given in Table 14.

It should be mentioned that the concrete cured with wet burlap was laid about the middle of October when the temperature was low. Therefore, the strength values reported cannot be considered as typical for that method of curing and serve only to indicate the degree of recovery from unsatisfactory curing in a period of 10 years.

Crack surveys of this project have been made from time to time with inter-

esting results. The slab lengths are given in Table 15.

The slab length for calcium chloride cured pavement decreased from 272 ft. to 165 ft. during the first 15 months, remained constant for the next 21 months and then decreased to 65 ft. in the next seven years. The age-slab-length curve for the earth cured pavement has a shape similar to that for calcium chloride curing. The slab length for sodium silicate curing decreased for 12 months from 168 ft. to 144 ft. and then remained constant for the next 45 months and then decreased to 92 ft. in the next 5 years.

TABLE 14

EFFECT OF VARIOUS METHODS OF CURING UPON COMPRESSIVE STRENGTH OF CORES

Hamilton County Project, Built in 1929

Method of curing	Compressive strength—lb. per sq. in.	
	Age 4 mo.	Age 10 yr.
Wet burlap and earth	(67) 5070	(5) 7834
	Percent of earth cured	Percent of earth cured
"Curcrete"	(6) 88.1	(5) 96.6
"Hunt Process"	(11) 99.8	(5) 95.6
$CaCl_2$—Surface	(11) 91.3	(5) 94.6
Sodium Silicate	(12) 80.7	(5) 90.9
Wet burlap	(12) 79.3	(5) 82.2

Figures in parentheses show number of specimens.

TABLE 15

Method of curing	1929		1939	
	Slab length, ft.	Percent of earth cured	Slab length, ft.	Percent of earth cured
Calcium chloride	271.1	131	65.5	144
Wet earth	207.8	100	45.6	100
Sodium silicate	168.3	81	92.3	202
Wet burlap	164.9	79	74.1	162
"Curcrete"	111.2	54	47.1	103
"Hunt process"	101.1	49	38.1	184

The slab length of pavement cured with bituminous coatings did not decrease appreciably from 2 to 57 months. The decrease in the next 5 years was at approximately the same rate as for the other methods of curing. These data are shown in Figure 1.

The relationship between slab length and strength of concrete on this project raises the question as to whether or not the methods of curing resulting in lower strengths and greater slab lengths may not have produced concrete having lower moduli of elasticity. It has been observed that concrete having a low modulus of elasticity develops transverse cracks at less frequent intervals than that having a higher value for this modulus.

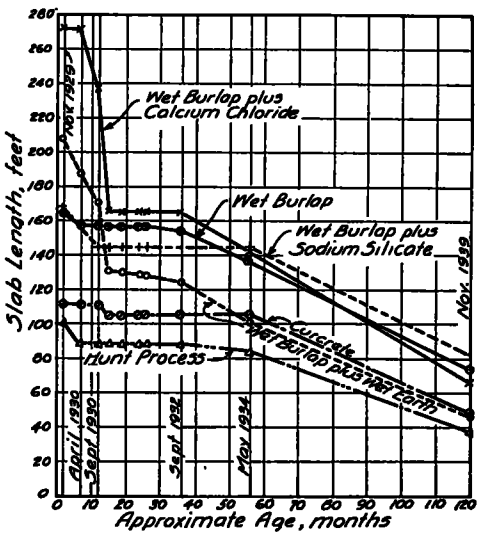


Figure 1. Effect of Kind of Curing on Rate of Change in Length of Slab. Built, Fall of 1929.

GENERAL CONCLUSIONS

The brief general conclusions that we have drawn from the data presented in this paper are:

1. Earth and water curing provides the most desirable results of any that we have studied.

2. The curing treatment applied during the first few hours after the concrete is placed has the greatest effect upon its strength.
3. The effect of a curing method upon strength may be predicted from laboratory tests for its prevention of moisture loss.
4. While under the climatic conditions existing in Iowa, differences of strength due to differences in curing methods tend to decrease with the age of the concrete, they do not disappear entirely for a very long time. The significance of the gain in strength with advanced age will depend upon whether or not the pavement survives the effect of stresses imposed upon it at early ages.
5. The differences in slab length due to differences in methods of curing tend to disappear with increasing age of the pavement.

A list of the men who should be given credit for assistance in this work would be very long and will not be included in its entirety. Special credit is due R. W. Crum, who directed the earlier part of the work; to Mark Morris, who directed a large part of the work; and to James W. Johnson, who has been connected with it throughout practically its whole history.

MARYLAND'S MOBILE SOILS LABORATORY

By George H. Dent
Maryland State Roads Commission

SYNOPSIS

After being in service well over a year, the trailer laboratory of the Maryland State Roads Commission has proved its value beyond question. Built at a cost of about $3200 on a standard 23-ft. chassis, the trailer includes a stove, oven, sink, light plant, compressor, sieve shaker, ventilating and drying fans, etc. The trailer is so equipped that all field tests may be made on soils, such as grading, density, moisture content, plasticity, organic matter, and others. While such determinations are not so accurate or so detailed as would be obtainable in the main laboratory, the results are much superior to what might be obtained in the usual field laboratory housed in some temporary field "shack." With this portable laboratory it is possible to make tests in an hour that would consume two days if test samples had to be submitted to the main laboratory.

The State Roads Commission of Maryland began soil studies in 1932 and since that time the personnel of the soils department has grown from one man to seven who devote their full time to soils work.

Since the beginning, the demand for quick results has been so great that in 1938 it was decided that much would be accomplished if a mobile laboratory could be set up that would be practical, economical, and efficient. With these thoughts in mind, bids were asked on a trailer laboratory of our own design. This trailer has been in service for about 14 months and it has surpassed our fondest dreams.

The need for such equipment was first apparent when stabilization work demanded frequent grading, density and moisture content determinations. A shelter or frame shack costing from $100 to $500, seldom level, or water-proof and never theft-proof was usually built to serve as a laboratory. These shacks were unhandy to move and each new project usually required a new one.

The trailer unit was built to fill this need. It was built on a standard 23-ft. chassis with interior of our own design. The total cost, including equipment, was $3,237. The trailer alone, inclusive of stationary equipment such as the stove, oven, sink, and light plant, etc. cost $2,185.

The laboratory unit is fitted with a self-contained electric light and power plant, but is also provided with an attachment so that commercial electricity may be used when obtainable. The electricity provides not only light, but operating power for such equipment as the sieve shaker, soil dispersion apparatus, ventilating and drying fans, etc. A running water supply of 36 gal. is forced through the system by compressed air. At present the air is furnished by a hand pump, but an electric compressor is to be installed in the near future.

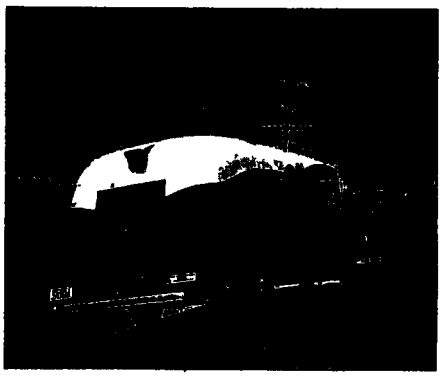


Figure 1. Maryland Mobile Soils Laboratory

A rigid plate holding a two-way level is mounted inside the trailer to insure

that all working surfaces are level. The leveling is accomplished by means of four jacks.

The oven and stove are gasoline fired and highly efficient, as well as compact. The heater uses kerosene for fuel.

The trailer contains equipment for the determination of the organic matter in topsoil by the Thomas Rapid Test Method. With this mobile laboratory, it is possible to go into a field, sample, test and give the results in less than an hour. This is a saving in time of at least two days.



Figure 2. Interior

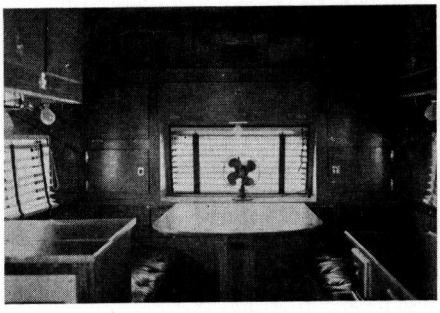


Figure 3. Office Section

Maryland is fortunate in having a background of test constants on some 3,000 soil samples. An experienced soils man, familiar with the geology and soil types of a territory and given such information, can in many cases, by a few simple operations, such as wash grading, or a quick plasticity index test, identify the material and determine its suitability.

The convenience of resampling encourages the securing of a more comprehensive picture of a source of supply. This added to the elimination of lost motion incidental to transporting samples, and the necessary correspondence, makes for a degree of efficiency and promptness seldom found in materials testing.

When unsuitable materials appear in gravel or borrow pits definite boundaries of acceptable materials can usually be set up by means of a series of tests. We do not attempt to obtain all the details or to attain the degree of accuracy given by the main laboratory. In many cases it is sufficient to find the sand, or clay content, the liquid limit or plasticity index. With this information, we offer the contractor the choice of the immediate approval of a material well within the specifications, or the necessary delay incidental to complete analysis by the Baltimore laboratory. The contractor has always welcomed the immediate approval which causes him no delay and results in our obtaining a superior product.

The trailer was first used in a gravel pit where difficulty in obtaining a uniform material was experienced. The laboratory moved on the job one afternoon. Twenty-four hours later, the material had been sampled, tests had been made, results tabulated, the contractor notified and construction work was again moving smoothly. In this particular case use of the Baltimore laboratory would have required a week to obtain the same results owing to the distance between the pit and the main laboratory.

In another instance, the borrowing of large quantities of soil for embankment was delayed due to the presence of some questionable soil. This project was more

than 100 miles from our main laboratory, but by moving the trailer to the spot, results were obtained in a few hours.

The field testing on a recent soil-cement project was carried on in the trailer. Owing to the type of soil and to the quantity of material larger than a No. 4 screen, the project was somewhat involved. It was necessary to make numerous screen analyses in order to obtain a void-free mixture. Frequent moisture-density determinations were made as well. These tests and others were made more accurately, economically and efficiently than would have been possible without the mobile laboratory.

Our trailer makes frequent trips to bituminous stabilization jobs to check the coarse and fine grading of materials, and to make any necessary adjustment in the mix. Density and depth checks are routine. A weekly or semi-weekly schedule has worked most satisfactorily on projects of this type.

Aside from the trouble shooting on excavation, borrow and gravel pits, and the constant use on stabilization work, we are finding the trailer laboratory indispensable on soil surveys in some localities.

Due to its efficient operation, from the viewpoint of both contractor and State, we feel the mobile laboratory is one of the best tools in a highway department's kit.

DEPARTMENT OF MAINTENANCE

W. H. Root, *Chairman*

REPORT OF COMMITTEE ON MAINTENANCE COSTS [1]

H. K. Bishop, *Chairman*

Chief, Division of Construction, Public Roads Administration

SYNOPSIS

A graphical presentation and review of maintenance cost trends on 18-ft. pavement widths in relation to traffic, for the region comprising Arkansas, Louisiana, Oklahoma and Texas. The following average annual surface maintenance costs per mile are given: Low-type pavements, ranging from $98 for traffic of 71 vehicles per 24-hour day to $577 for 484 vehicles. Intermediate type, from $186 for 91 vehicles to $249 for 1163 vehicles. High type, from $64 for 272 vehicles to $40 for 5312 vehicles.

The following are field inspection ratings of surface maintenance associated with above costs: Low-type surface 83, Intermediate 87, High 89. Numerical deductions from rating of 100 represent extent of repairs needed. When related to construction costs the above maintenance expenditures, within traffic limits shown on graphs, are not only economical but there is margin of additional expenditures that may be made before reconstruction to higher type is to be considered. The data do not consider savings in motor vehicle operating costs which would change latter relations.

During the last two meetings we called to your attention the variations that exist in annual highway maintenance cost and the necessity for accumulating and studying averages over a period of years. Our reports were published in the Highway Research Board *Proceedings*, Vol. 17, pages 384 to 388, Vol. 18, pages 298 to 304.

The study is being conducted through the facilities of the Public Roads Administration and covers 1,233 separate highway sections in 47 States and extends over 18,716 miles of the principal types of surfaces in use. This includes the 622 miles of maintenance sections in Connecticut, New Hampshire and Rhode Island for which another committee of the Highway Research Board is securing a more detailed traffic count.

Our attention was at first devoted to securing a uniform reporting of maintenance costs, each of the 47 States participating in the study at that time having its own and unrelated system of defining and recording this type of information. The problem has now been solved through the common use of B.P.R. Form M-1 originating with this committee during the organizing stage of the study in 1933, and partly rearranged in 1938 for coordination with the work of the committees on Uniform Accounting of the American Association of State Highway Officials and Highway Research Board. In view of the success of the forms, copies are attached to this report for possible use of other committees engaged in similar studies.

We have maintenance costs accumulated for a 4-year period and segregated for the following six highway elements, surface, shoulders, drainage, structure repairs, roadside and traffic service, which are reported under the following definition of maintenance: "General highway maintenance is the function of preserving

[1] This study is being conducted under the auspices of the Public Roads Administration by the Construction Division, Mr. H. K. Bishop, chief. Acknowledgment is made of the work by Mr. R. F. Severs, in charge of maintenance, and Mr. H. A. Radzikowski on the study and this report. Computations by Messrs. W. T. Hughes and J. T. Dressel.

B.P.R. Maintenance Cost Form M-1

REPORT OF ANNUAL COST OF
ON SECTIONS SELECTE

State .....

Section number ..... Location ..... Type surfacing .....

Type shoulders ..... Width shoulders ..... Type sho

Traffic count: Trucks ..... Cars ..... Estimated life of surfacing ..... Federal-a

Additional description of section .....

DEFINITION OF MAINTENANCE.—General highway maintenance is the function of preserving and keeping each ty produce satisfactory service. General maintenance does not include reconstruction or addition and betterments.

ANNUAL MA

SURFACING	PATCHING	JOINT AND CRACK FILLING	DRAGGING	BASE AND SUB-BASE REPAIRS	RETREATMENTS AND ALL CaCl2	
SHOULDERS	PATCHING	DRAGGING	SODDING	RETREATMENTS		
DRAINAGE	DITCHES	DRAINS	CULVERT CLEANING			
STRUCTURE REPAIR (Including bridges not over 20-ft. span)	BRIDGE AND CULVERT	GUARD RAIL				
ROADSIDE	GRASS AND WEED CUTTING	REMOVAL OF DEBRIS	HIGHWAY BEAUTIFICATION	CUTS	FILLS	
TRAFFIC SERVICE	HIGHWAY MARKERS	GUIDE-LINE PAINTING	SNOW REMOVAL			

GENERA

1. ARE ALL RECONSTRUCTION CHARGES EXCLUDED FROM ABOVE COSTS? .....

2. WHICH OF THE ABOVE COSTS WERE OBTAINED BY PRORATING? .....

METHOD OF CALCULATING EQUIPMENT DEPRECIATION .....

LIST MAJOR EQUIPMENT USED ON SECTION, WITH DEPRECIATION RATES .....

.....

AVERAGE COST OF MAJOR LABOR ITEMS .....

AVERAGE COST OF MAJOR MATERIAL ITEMS .....

ORIGINAL CONSTRUCTION COST		RECONSTRUCTION COSTS SINCE CONSTRUCTION	ADDITIO (I
Total	Surface		

Additional pertinent information:

U.S. GOVERNM

RAL HIGHWAY MAINTENANCE
SPECIAL COST STUDY

Report for year ending .....

..... Length ..... Width ..... Thickness ..... Year built .....

ment ..... Shoulder width treated .....

included ..... Maintenance quality rating (B.P.R.).....

.....

way, structure, and facilities as near as possible in their original condition as constructed, or as subsequently improved, to

.....

ANCE COSTS

HER STATE CLASSIFICATIONS			TOTALS	LABOR	MATERIALS	EQUIPMENT	OVERHEAD
		GRAND TOTALS,					

ESTIONS

3. ARE FIELD AND OFFICE OVERHEAD COSTS INCLUDED IN ABOVE COSTS? .....

4. ARE ALL EQUIPMENT DEPRECIATION, OPERATING, AND OVERHEAD COSTS INCLUDED? .....

.....

.....

.....

.....

ETTERMENT CHARGES urface treatment)	REMARKS AS TO UNUSUAL CONDITIONS AFFECTING MAINTENANCE COSTS (TOPOGRAPHY, INADEQUATE DESIGN, SOIL, TRAFFIC WEIGHT, CLIMATE, EXTRAORDINARY EMERGENCY WORK, EXCESSIVE HAUL AFFECTING TRANSPORTATION COSTS, ETC.)

S OFFICE: 1934

and keeping each type of roadway, structures and facilities as nearly as possible in original condition as constructed or subsequently improved to produce satisfactory service." Our remaining problem is to present the data in form that will not be misused or misconstrued.

Due to the importance of the element of surface, it is being singled out in this report and a tentative analysis submitted in graphical form for our West South Central climatic region. The area is comprised of the States of Arkansas, Louisiana, Oklahoma and Texas. It is hoped that after time has been allowed for review of this presentation it will stimulate suggestions that may be helpful in drafting our final report.

The data in Figures 1 and 2 are confined to surfaces 18 ft. wide, segregated under three general classifications as to surface type—high, intermediate and low [1]—and based on Form M-1 records for a 4-year period. The graphs show the best fitted straight-line trend as determined by the theory of least squares, for all values plotted on work sheets. The following number of values determined the trends: High 51, Intermediate 26, and Low 11—covering a total of 1,282 miles. The lines are shown only within the limits of actual traffic count plotted and are not extended for theoretical values. The traffic is a weighted average 24-hour count for all classes of vehicles using the sections.

Figure 1 represents a trend of annual average surface maintenance cost per mile plotted against traffic.

[1] General classification of pavements:
High-type surfaces: concrete, brick, bituminous concrete—rigid and non-rigid base, bituminous macadam.
Intermediate-type surfaces: oil processed, surface treated stone, gravel, sand-clay, etc.
Low-type surfaces: untreated stone, gravel, sand-clay, graded.

Figure 2 represents a trend of average total annual surface cost per mile plotted against traffic.

The total annual surface cost is calculated by adding together the average annual surface maintenance cost and an annual surface construction charge based on life of surface reported from actual field inspections. No charge was made for interest on construction investment as none was reported expended on the sections. The construction item includes only surface and base costs. Grading and other highway costs were considered as fluctuating too widely with local conditions. A comparison of the economy of maintenance on various types of pavements must therefore start from finished grading.

A third graph suggests itself, the surface costs as shown in Figure 2 plus a charge to compensate for savings in motor-vehicle operating costs over the various types of surface, referred to in a report by Mr. R. A. Moyer in Vol. 18, Highway Research Board *Proceedings*, pages 41 to 60. The data therein covered three types of surface while our reports refer to three general classifications of the 57 types of surface included in the study. It being considered outside the scope of the work of the maintenance cost committee to attempt to secure more extensive data on automobile expenditures, no study is being made of the relation of surface maintenance costs to total annual surface charges combined with savings in motor-vehicle operating costs.

Turning to Figure 1, it will be observed that the trend of surface maintenance cost per mile is the highest for the low-type group above traffic of 140 vehicles per 24-hour day, less for intermediate types, and lowest for high types. These trends are not, however, indicative as to the economy of expenditures for reasons to be shown presently in Figure 2.

Figure 1 further shows a steep upward

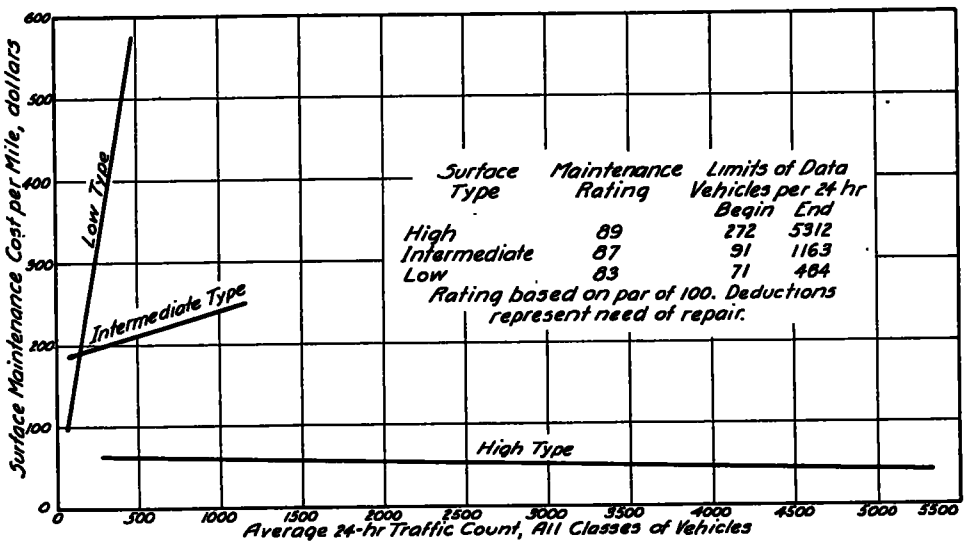


Figure 1. Total Annual Surface Maintenance Cost per Mile Trends by Average 24-hr. Traffic Count. 18 ft. Width Surfaces in West-South-Central Region. Based on Data for Four Years, 88 Sections, 1282 Miles.

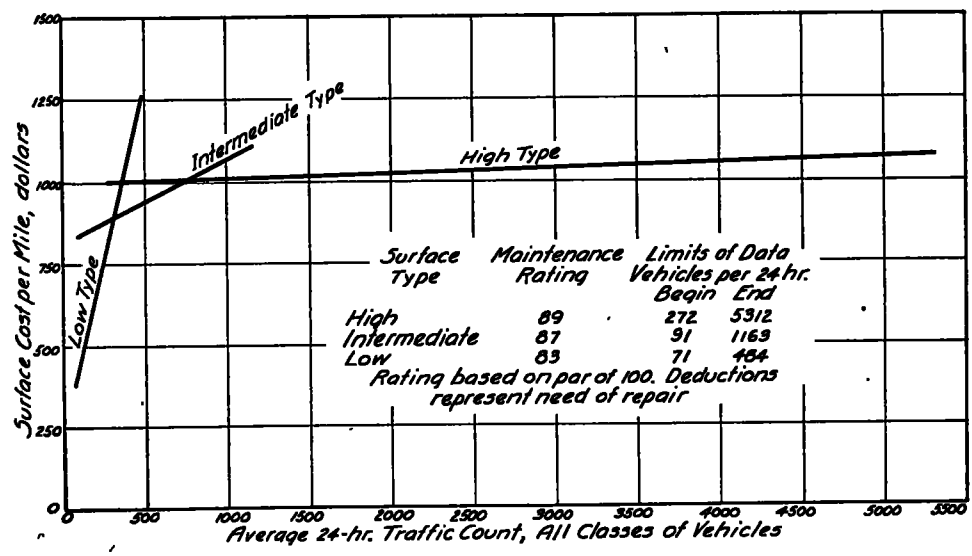


Figure 2. Total Annual Surface Cost per Mile Trends by Average 24-hr. Traffic Count. 18 ft. Width Surfaces in West-South-Central Region. Based on Data for Four Years, 88 Sections, 1282 Miles.

B. P. R. Maintenance Section Cost Form M-1 (Revised)

REPORT OF ANNUAL COST OF
ON SECTIONS SELECTED

State _____

Section number _____ Location _____ Type surfacing in code ___

Type shoulders _____ Width shoulders _____ Ty

Traffic count: Trucks _____ Cars _____ Estimated future life of surfacing _____

Additional description of section _____

DEFINITION OF MAINTENANCE.—Highway maintenance is the preserving and keeping of each type of roadway, roadside, str facilities and services to provide satisfactory and safe highway transportation. Maintenance does not include reconstruction or addit

ANNUAL MAIN
(Subdivision by operations opti

MAIN CLASSIFICATIONS	1	2	3	4	5	
SURFACE-ROUTINE	Patching	Dragging, etc.	Joints and cracks			
SURFACE-SPECIAL	Dust palliatives	Replacements	Reprocessing	Bit. treatment	Mud-jacking	Traffic
SHOULDERS AND APPROACHES	Patching, etc.	Reseeding, resodding	Ribbon bit. tr.	Retreatments		
ROADSIDE AND DRAINAGE	Cuts, fills, washouts	Drainage channels	Roadside cleaning	Roadside development	Misc. structures	
TRAFFIC SERVICES	Signs, etc.	Surface markings	Guard rail	Lighting, electricity	Comfort stas. etc.	D
SNOW, ICE, AND SAND CONTROL	Snow fence	Snow removal	Sanding	Opening waterways	Sand drifts	
STRUCTURES (Over 20 ft.)						
EXTRAORDINARY REPAIRS						
GENERAL EXPENSE	Office	Field supervision				

GENERAL

1 ARE ALL RECONSTRUCTION CHARGES EXCLUDED FROM ABOVE COSTS? _____

2. WHICH OF THE ABOVE COSTS WERE OBTAINED BY PRORATING? _____

METHOD OF CALCULATING EQUIPMENT DEPRECIATION _____

LIST MAJOR EQUIPMENT USED ON SECTION, WITH DEPRECIATION RATES _____

AVERAGE COST OF MAJOR LABOR ITEMS _____

AVERAGE COST OF MAJOR MATERIAL ITEMS _____

ORIGINAL CONSTRUCTION COST		RECONSTRUCTION COSTS SINCE CONSTRUCTION	ADDITION A (Als
Total	Surface		

Additional pertinent information:

U S GOVERNMENT PRI

ENERAL HIGHWAY MAINTENANCE
OR SPECIAL COST STUDY

Report for year ending _____

_____ Length _____ Width _____ Thickness _____ Year built _____

shoulder treatment _____ Shoulder width treated _____

Federal-aid projects included _____ Maintenance quality rating (B. P. R.) _____

ure, and facility as nearly as possible in its original condition as constructed or as subsequently improved, and the operation of highway s and betterments.

ENANCE COSTS
but desirable if kept by the State)

	OTHER STATE CLASSIFICATIONS			TOTALS	LABOR	MATERIALS	EQUIPMENT
tection							
ure							
			GRAND TOTALS,				

QUESTIONS

3. ARE FIELD AND OFFICE OVERHEAD COSTS INCLUDED IN ABOVE COSTS? _____

4. ARE ALL EQUIPMENT DEPRECIATION, OPERATING, AND OVERHEAD COSTS INCLUDED? _____

ND BETTERMENT CHARGES first surface treatment)	REMARKS AS TO UNUSUAL CONDITIONS AFFECTING MAINTENANCE COSTS (TOPOGRAPHY, INADEQUATE DESIGN, SOIL, TRAFFIC WEIGHT, CLIMATE, EXTRAORDINARY EMERGENCY WORK, EXCESSIVE HAUL AFFECTING TRANSPORTATION COSTS, ETC.)

NTING OFFICE 8—10883

slope on the low-type maintenance cost line for increasing quantities of surface use. Starting with an annual maintenance cost of $98 per mile for 71 vehicles per 24-hour day and rising to a cost of $577 for a traffic of 484 vehicles. The intermediate type also shows an upward trend with quantity of traffic use although much less steep, starting with an annual cost of $186 for 91 vehicles per day and ending with $249 for 1,163 vehicles.

On the other hand the high-type maintenance cost line shows a slightly downward trend from $64 per mile for a count of 272 vehicles per day to $40 per mile at the 5,312 vehicle point. Although this was contrary to expectations, investigation indicated that construction costs were higher at the lower maintenance cost end of the line. From this it may be concluded that with anticipated increase in traffic use durability was built into the pavement and maintenance costs went down. This will be made more apparent from Figure 2 where the high-type line for combined annual maintenance and construction costs slopes slightly upward with increased traffic and from reports of extended life of surface.

Based on a par of 100, the average field inspection ratings of surface maintenance associated with above costs were: Low-type 83, intermediate 87, and high 89. The numerical deductions from the standard represent the extent of repairs the surfaces needed.

Figure 2 discloses a relation, in part the reverse of that shown in Figure 1. The maintenance costs on the graph, when related to construction charges by combination, are shown to be within economic limits for:

(a) Low-type surfaces, from the traffic value of 71 to 315 vehicles.
(b) Intermediate types, from the traffic value of 315 to 767 vehicles.
(c) High types, from the traffic value of 767 to 5,312 vehicles, the end point of our data.

The 315 and 767 traffic values are determined by the intersections of the low-intermediate type and intermediate-high type annual surface cost trend lines. Transferring the traffic data to Figure 1, the maintenance costs within economic limits are shown to range for:

(a) Low-type surfaces, from $98 to $381 per mile.
(b) Intermediate-type surfaces, from $199 to $226 per mile.
(c) High-type surfaces, from $62 to $40 per mile.

These costs are not only economical but the cost trends in Figure 2 show, within a limited range of traffic values, that an additional margin of maintenance expenditures can be made before reconstruction to higher surface type should be considered. The cost trends at the 200 and 400 traffic values are cited as examples. The gap at the 200 traffic point represents that $221 more per mile per year can be spent for maintenance before the low-type surface becomes uneconomical and reconstruction to an intermediate type should be considered on a cost basis. A similar analysis at the 400 traffic point of the graph shows a margin of $88 more per mile per year available for maintenance before the intermediate type approaches a reconstruction stage. It seems that the field inspection ratings cited above indicate that at least part of this money could have been used to bring the maintenance up nearer to the par of 100.

The graphical relations shown will vary for each of the nine regions into which the country was divided for the study, depending on climatic, topographical and other conditions. We hope to have at the next meeting similar information to that presented here for each

region, covering a maintenance cost average for five years and a summary for the entire United States.

CONCLUSIONS

1. Annual average surface maintenance cost straight-line trend, based on records for 4-year period, for low-type pavements, 18 ft. wide on highway sections in area mentioned in report, ranged from $91 per mile for a surface use of 71 vehicles per 24-hour day to $577 for a traffic of 484 vehicles.

2. For intermediate-type pavements the range is from $186 per mile for 91 vehicles per day to $249 for 1,163 vehicles.

3. For high-type pavements the range is from $64 per mile for 272 vehicles to $40 for 5,312 vehicles. Additional durability built into the pavements at higher traffic values lowered the maintenance costs.

4. A measure as to whether above maintenance expenditures are within economic limits is available by comparing the total annual surface cost per mile for each of the three types. This cost is a combined average annual maintenance and surface construction charge.

5. Graphical analysis of the total annual surface cost indicates, within certain traffic values, that these maintenance costs are not only within economic limits, but that additional maintenance expenditures could be made before reconstruction to a higher surface type should be considered on a cost basis. Field maintenance inspection ratings reflected that part of this expenditure would have been desirable to improve condition of surface.

PROGRESS REPORT ON MAINTENANCE COST STUDIES IN RELATION TO TRAFFIC

By A. H. Worley

Associate Highway Engineer, Public Roads Administration

SYNOPSIS

In cooperation with the States of Connecticut, New Hampshire and Rhode Island, the Public Roads Administration is conducting studies of the maintenance cost of different surface types as related to the traffic carried. The study, which began in the spring of 1935 and will cover a period of 5 years, includes 178 sections totaling 622 miles.

The studies are divided into three parts: traffic density, inventory and analysis of maintenance costs. Traffic densities in each State are being determined at 44 stations, with counts being staggered in such fashion as to provide a basis for computing the average 24-hr. traffic for each year. Weights of the various vehicles using the highways will be furnished by the Highway Planning Surveys. A physical inventory of the sections was necessary to determine the items entering into the maintenance cost record. This included descriptions of the surface, base and sub-base, shoulders, culverts, bridges, guard rail, traffic services, etc. Maintenance costs are reported on selected units of measure for six sections: surfacing, shoulders, drainage, structure repair, roadside cleaning, and traffic services.

The Public Roads Administration is cooperating with the Highway Departments of the States of Connecticut and New Hampshire and the Department of Public Works of the State of Rhode Island in making studies of the cost of general highway maintenance on certain selected sections of highway having different types of surfacing and relating the cost of maintaining the surfacing with the traffic.

The studies were begun in the spring of 1935 to continue for a period of five years.

The types of surfacing, number and length of sections for each type on the selected sections are given in Table 1.

The sections are located throughout the States of Connecticut and Rhode Island and the southern half of New Hampshire as indicated on Figure 1—a map of Connecticut, Figure 2—a map of Rhode Island, and Figure 3—a map of New Hampshire.

For convenience, the studies are divided into three separate parts, namely, traffic density, inventory, and analysis of maintenance costs. The first part is a determination of the amount of traffic that traverses each of the various sections, the second part is the inventory of the various parts of the highway for which annual maintenance costs are reported, and the third part is the analysis of the maintenance cost data in such a manner that comparisons may be easily and readily made at the end of the five-year period.

The traffic density is being obtained by counting the traffic at 44 main stations in each State. The 44 stations are divided into two groups, of 22 stations each. Each group is operated by one man according to a fixed schedule during periods of 26 days each which permits him to be off duty four days during the 26. The stations are operated from 6 a.m. to 2 p.m. and from 2 p.m. to 10 p.m. during alternate periods, thereby obtaining a count at each station on each day of the week for these hours in each year. The stations are also operated from 10 p.m. to 6 a.m. twice during each year. The average 24-hour traffic for each year is computed from the amount of traffic counted during these operations.

A record of the traffic counted on each section of highway during each operation is submitted to the Division of Highway Transport on a form which shows any factors affecting the normal flow of traffic on the date of operation, also all maintenance, and changes in the condition of the surfacing, since the last operation of the station which was 26 days previous. The data on these record forms for each section are posted by years on B.P.R. Maintenance Cost Form MT-1 as shown on Table 2. It is to be noted that this form shows the dates, days of the week, and hours of the day when traffic was counted; the classification of the vehicles; the averages for the hourly periods and the average 24-hour traffic; also that the elapsed time between the counting of traffic in the morning and the afternoon on the same day of the week was six months.

For further study of the traffic density, the average 24-hour traffic on each section for each year is copied on a form, Table 3, especially prepared for the purpose.

Table 4 shows the summary of the 24-hour average number of vehicles passing the traffic stations during each year that the traffic has been counted, and the trends compared with the first year. The traffic at stations in Connecticut affected by the opening of the Merritt Parkway is not included in this tabulation.

In order to make a comparison of the costs of maintenance for the various items, it is necessary to have a physical inventory of each section.

The description of the surfacing including the base and sub-base has been furnished by the States from construction records. The inventory had to be obtained in the field for other items such as

TABLE 1

State	Type of surfacing	Number of sections	Length in miles
New Hampshire	Concrete	17	64.238
	Modified asphalt	4	12.173
	Bituminous macadam	13	39.891
	Surface treated gravel	15	90.798
	Gravel (untreated)	4	8.151
	Total	53	215.251
Rhode Island	Concrete	21	86.945
	Asphalt	14	42.884
	Bituminous macadam	27	83.461
	Waterbound macadam, surface treated	11	24.179
	Surface treated gravel	19	51.420
	Total	92	288.889
Connecticut	Concrete	7	27.181
	Asphalt	6	17.018
	Bituminous macadam	7	23.070
	Waterbound macadam, surface treated	6	26.625
	Surface treated gravel	7	23.596
	Total	33	117.490
	Total for all states	178	621.630

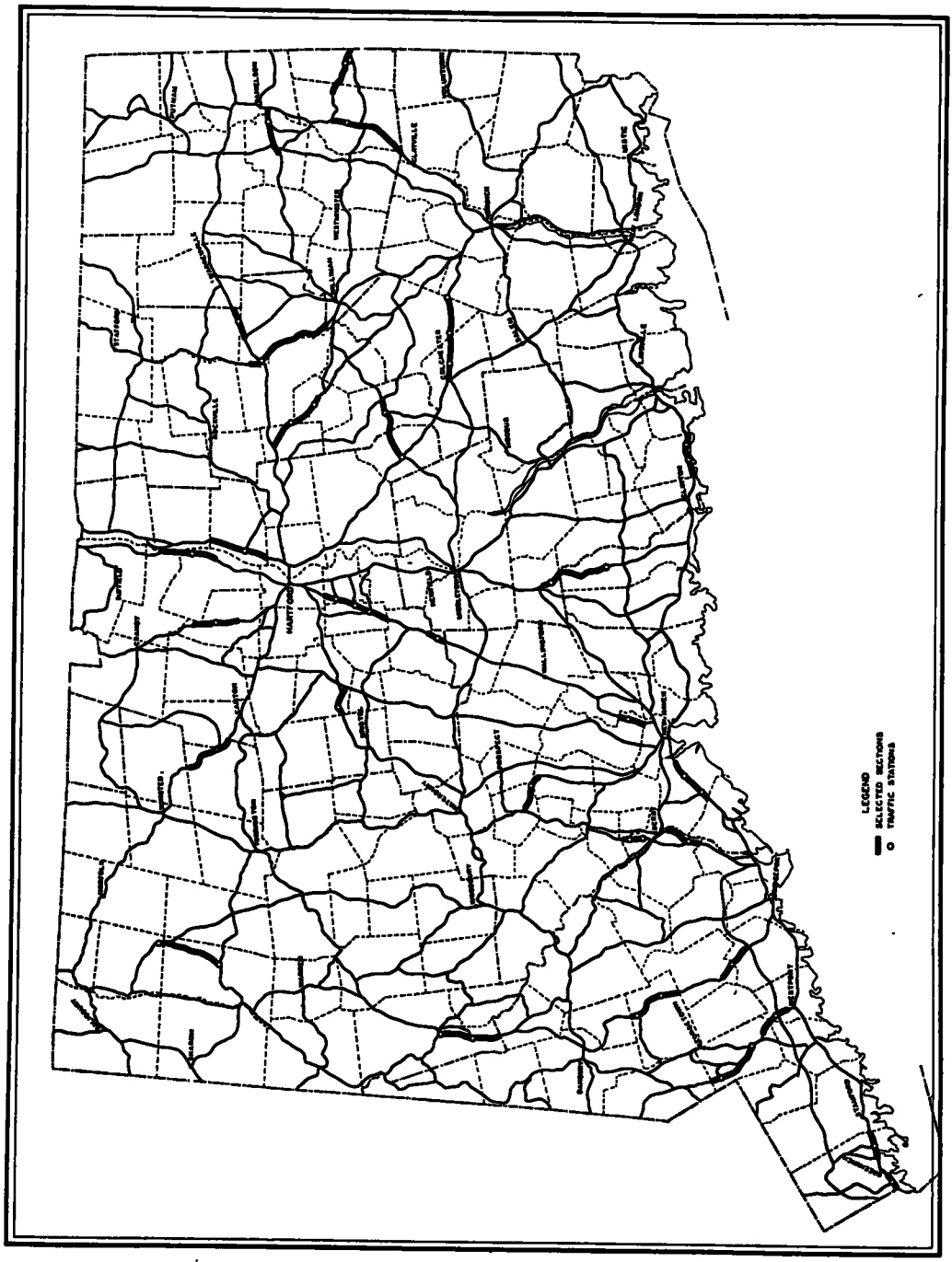


Figure 1. Connecticut Highways, Maintenance Cost Study Sections

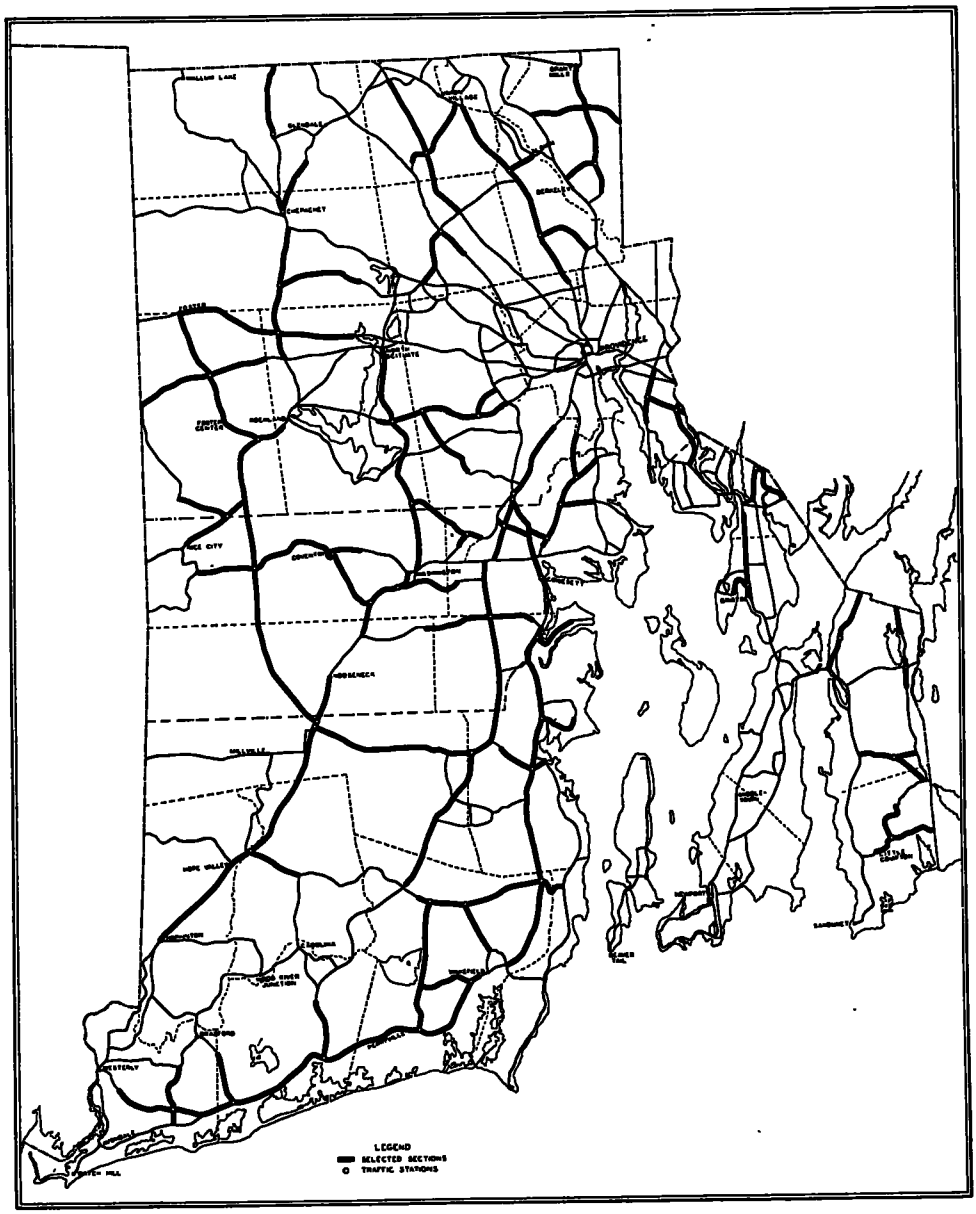


Figure 2. Rhode Island Highways, Maintenance Cost Study Sections

width of treated and untreated shoulders; size, kind, and length of culverts; type and length of guard rail; description and amount of roadside cleaning; number and kind of highway markers; and the corrugated metal pipe, reinforced concrete pipe, cast iron pipe, or vitrified clay pipe. The side drains for private drives entering the highway are practically all corrugated metal pipe. The

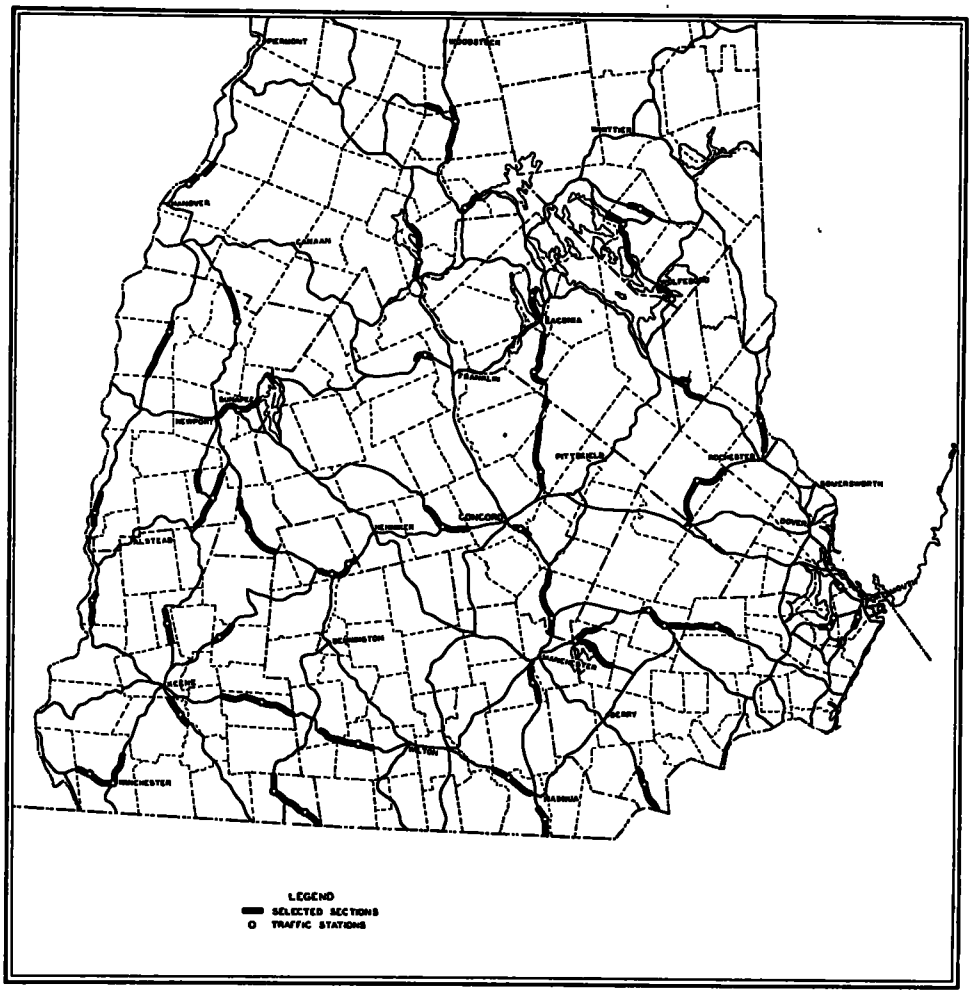


Figure 3. New Hampshire Highways, Maintenance Cost Study Sections

length of guide line painting except in New Hampshire where a record is kept by the paint department. For some of the sections the data for culverts were copied from construction plans.

The field work disclosed that transverse culverts on a section are principally guard rail in Connecticut is cable on wooden posts for practically all sections; in Rhode Island it is almost without exception a wooden rail type; and in New Hampshire cables on wooden posts, cables on concrete posts, the wooden rail type, and during recent years posts with-

TABLE 2

MAINTENANCE COST STUDIES

Traffic counted on		Remarks	Pass. cars	Trucks*						Busses			Total vehicles
Date	Day	Weather—Extensive repairs—New work—Other circumstances affecting traffic		1½ tons and under	Over 1½ tons, under 5 tons		5 tons and over		Total	25 pass. or less	More than 25 pass.	Total	
					Type A	Type B	Type A	Type B					
HOURS 6 A. M. To 2 P. M.													
5–2–38	Monday	Clear	1,967	113	169	5	6	54	347	7	9	16	2,330
6–23–38	Thursday	Clear	2,072	150	167	8	8	55	388	3	10	13	2,473
8–21–38	Sunday	Clear	5,280	84	39	..	..	10	133	3	26	29	5,442
10–19–38	Wednesday	Clear	1,646	126	117	7	14	59	323	6	12	18	1,987
12–10–38	Saturday	Rain	1,314	80	105	6	5	47	243	3	10	13	1,570
1–31–39	Tuesday	Snow	558	66	108	3	10	59	246	4	8	12	816
3–24–39	Friday	Clear	1,174	106	107	6	10	59	288	7	9	16	1,478
Week-day average (A)			1,483	112	133	6	10	57	318	5	10	15	1,816
Saturday average (B)			1,314	80	105	6	5	47	243	3	10	13	1,570
Sunday average (C)			5,280	84	39	..	..	10	133	3	26	29	5,442
Total average (5A+B+C)÷7			2,001	104	116	5	7	49	281	5	12	17	2,299
HOURS 2 P. M. To 10 P. M.													
4–6–38	Wednesday	Clear entire day	1,529	89	113	5	14	77	298	3	15	18	1,845
5–28–38	Saturday	Clear entire day	5,640	129	102	4	11	53	299	5	32	37	5,976
7–26–38	Tuesday	Clear entire day	3,193	103	128	4	15	86	336	7	19	26	3,555
9–23–38	Friday	Clear entire day	3,141	130	135	5	12	74	356	6	16	22	3,519
11–14–38	Monday	Clear entire day	1,563	77	123	6	12	69	287	3	17	20	1,870
1–5–39	Thursday	Clear entire day	1,081	64	77	2	5	36	184	4	12	16	1,281
2–26–39	Sunday	Rain entire day	2,103	57	17	..	2	20	96	3	13	16	2,215
Week-day average (A)			2,102	93	115	4	12	68	292	4	16	20	2,414
Saturday average (B)			5,640	129	102	4	11	53	299	5	32	37	5,976
Sunday average (C)			2,103	57	17	..	2	20	96	3	13	16	2,215
Total average (5A+B+C)÷7			2,607	93	99	4	10	59	265	4	18	22	2,894

TABLE 2—Continued

Traffic counted on		Remarks	Pass. cars	Trucks*						Busses			Total vehicles
Date	Day	Weather—Extensive repairs—New work—Other circumstances affecting traffic		1½ tons and under	Over 1½ tons, under 5 tons		5 tons and over		Total	25 pass. or less	More than 25 pass.	Total	
					Type A	Type B	Type A	Type B					
Hours 10 P. M. To 6 A. M.													
6-23-38	Thursday....	Clear.....	469	29	76	2	4	129	240	..	3	3	712
12-10-38	Saturday....	Clear 10 p. m. to 4.30 a. m. Rain 4.30 a. m. to 6 a. m...	203	8	23	16	17	41	105	..	4	4	312
Week-day average (A).....			406	20	56	1	6	147	230	..	4	4	640
Saturday average (B).....			537	10	23	13	27	44	117	..	8	8	662
Sunday average (C).....			570	7	6	..	3	13	29	..	8	8	607
Total average (5A+B+C)÷7			448	17	44	3	8	113	185	..	5	5	638
Summary													
Total average 6 a. m. to 2 p. m.....			2,001	104	116	5	7	49	281	5	12	17	2,299
Total average 2 p. m. to 10 p. m.....			2,607	93	99	4	10	59	265	4	18	22	2,894
Total average 10 p. m. to 6 a. m.....			448	17	44	3	8	113	185	..	5	5	638
Average 24-Hour Traffic.....			5,056	214	259	12	25	221	731	9	35	44	5,831

* Type A—Single vehicles with two axles. Type B—Single vehicles with three axles and tractor-semitrailers with three or more axles.

TABLE 3

MAINTENANCE COST STUDIES
Average Twenty-Four Hour Traffic

Station No.	Maintenance cost studies section						Number of pass. cars	Number of trucks*						Number of buses			Total number of vehicles
	Number	On route No. or highway	Direction from station	Toward	Length (miles)	Total length of section		1½ tons and under	Over 1½ tons, under 5 tons		5 tons and over		Total	25 pass. or less	More than 25 pass.	Total	
									Type A	Type B	Type A	Type B					
	(1) Traffic census for year ending April 19, 1936.					(2) Traffic census for year ending April 18, 1937.											
	(3) Traffic census for year ending April 17, 1938.					(4) Traffic census for year ending April 30, 1939.											
344	75	U. S. 1	NE and SW	Stamford and Port Chester	....	(1)	16,123	1,170	631	95	500	445	2,841	27	165	192	19,156
						(2)	18,028	1,197	618	258	512	542	3,127	17	164	181	21,336
						(3)	17,374	1,093	467	155	514	612	2,841	5	126	131	20,346
						(4)	12,894	1,148	391	129	474	629	2,771	7	114	121	15,785
307	76	U. S. 6A	NW and SE	Bolton Notch and Andover	....	(1)	1,418	113	46	4	32	15	210	25	4	29	1,657
						(2)	1,452	125	40	2	32	12	211	19	7	26	1,689
						(3)	2,070	153	37	5	33	27	255	22	3	25	2,350
						(4)	2,300	147	41	5	33	27	252	14	2	16	2,568
308	76	U. S. 6A	N.W.	Bolton Notch	....	(1)	1,482	130	66	6	33	13	248	28	4	32	1,762
						(2)	1,551	145	54	4	35	17	255	23	6	29	1,835
						(3)	1,996	143	41	3	35	22	244	19	3	22	2,262
						(4)	1,723	137	34	2	31	25	229	13	3	16	1,968
			S.E.	Willimantic	....	(1)	1,513	135	65	8	35	13	256	30	4	34	1,803
						(2)	1,559	157	55	4	49	17	282	22	6	28	1,869
						(3)	1,975	143	41	3	35	22	244	19	3	22	2,241
						(4)	1,710	137	34	2	31	25	229	13	3	16	1,955
305	77	U. S. 5A	North	Windsor Locks	....	(1)	3,483	290	178	49	146	94	757	86	52	138	4,378
						(2)	3,790	337	153	60	155	170	875	47	55	102	4,767
						(3)	4,785	326	130	44	97	126	723	9	55	64	5,572
						(4)	4,380	344	113	30	113	153	753	14	43	57	5,190
			South	Windsor	....	(1)	3,658	323	180	49	147	93	792	90	53	143	4,593
						(2)	3,984	393	160	60	160	172	945	48	57	105	5,034
						(3)	5,026	373	136	43	99	111	762	10	58	68	5,856
						(4)	4,655	388	117	30	114	151	800	14	47	61	5,516

* TYPE A—Single vehicles with two axles. TYPE B—Single vehicles with three axles and tractor-semitrailers with three or more axles.

TABLE 4

MAINTENANCE COST STUDIES

Summary of the 24-Hour Average Number of Vehicles Passing the Traffic Stations

State	Year ending	Passenger cars	Trucks						Busses			Total vehicles
			1½ tons and under	Over 1½ tons under 5 tons		5 tons and over		Total	25 passenger or less	More than 25 passenger	Total	
				Type A	Type B	Type A	Type B					
		Number										
New Hampshire.....	3-29-1936.....	64,409	5,085	3,801	305	321	405	9,917	202	306	508	74,834
	3-28-1937.....	72,060	5,928	4,212	348	408	768	11,664	254	349	603	84,327
	3-27-1938.....	77,087	5,829	4,146	266	342	795	11,378	225	317	542	89,007
	4-10-1939.....	72,196	5,859	4,259	231	304	999	11,652	223	291	514	84,362
		Percent										
	3-29-1936.....	100	100	100	100	100	100	100	100	100	100	100
	3-28-1937.....	112	117	111	114	127	189	118	126	114	119	113
	3-27-1938.....	120	115	109	87	107	196	115	111	104	107	119
	4-10-1939.....	112	115	112	76	95	247	117	110	95	101	113
		Number										
Connecticut.....	4-19-1936.....	97,684	8,714	4,141	789	2,988	2,660	19,292	548	1,294	1,842	118,818
	4-18-1937.....	110,950	9,657	3,797	1,263	3,446	3,534	21,697	431	1,159	1,590	134,237
	4-17-1938.....	122,510	9,427	3,274	946	3,207	3,259	20,113	385	1,049	1,434	144,057
	4-30-1939.....	119,939	9,641	2,975	933	3,108	3,378	20,035	341	1,025	1,366	141,340
		Percent										
	4-19-1936.....	100	100	100	100	100	100	100	100	100	100	100
	4-18-1937.....	114	111	92	160	115	133	112	78	90	86	113
	4-17-1938.....	125	108	79	120	107	123	104	70	81	78	121
	4-30-1939.....	123	111	72	118	104	127	104	62	79	74	119

TABLE 4—Continued

State	Year ending	Passenger cars	Trucks						Busses			Total vehicles
			1½ tons and under	Over 1½ tons under 5 tons		5 tons and over		Total	25 passenger or less	More than 25 passenger	Total	
				Type A	Type B	Type A	Type B					
		Number										
Rhode Island.....	7-18-1936.....	124,007	12,800	3,131	511	1,677	645	18,764	422	1,181	1,603	144,374
	7-17-1937.....	140,346	13,815	3,529	590	1,592	908	20,434	481	1,155	1,636	162,416
	6-21-1938.....	150,271	13,260	3,136	444	1,414	951	19,205	403	1,126	1,529	171,005
	7-11-1939.....	155,689	14,843	3,277	481	1,429	1,158	21,188	379	1,218	1,597	178,474
		Percent										
	7-18-1936.....	100	100	100	100	100	100	100	100	100	100	100
	7-17-1937.....	113	108	113	115	95	141	109	114	98	102	113
	6-21-1938.....	121	104	100	87	84	147	102	95	96	95	118
	7-11-1939.....	125	116	105	94	85	180	113	90	103	100	124
		Number										
All states.....	First year.....	286,100	26,599	11,073	1,605	4,986	3,710	47,973	1,172	2,781	3,953	338,026
	Second year.....	323,356	29,400	11,538	2,201	5,446	5,210	53,795	1,166	2,663	3,829	380,980
	Third year.....	349,868	28,516	10,556	1,656	4,963	5,005	50,696	1,013	2,492	3,505	404,069
	Fourth year	347,824	30,343	10,511	1,645	4,841	5,535	52,875	943	2,534	3,477	404,176
		Percent										
	First year.....	100	100	100	100	100	100	100	100	100	100	100
	Second year....	113	111	104	137	109	140	112	99	96	97	113
	Third year.....	122	107	95	103	100	135	106	86	90	89	120
	Fourth year....	122	114	95	102	97	149	110	80	91	88	120

NOTE: TYPE A—Single vehicles with two axles.
TYPE B—Single vehicles with three axles and tractor semi-trailers with three or more axles.

out cables or wooden rails have been erected. The kind and type of highway markers are varied; however, each State erects curve, keep single line, and side road or road intersection signs which indicate the alignment and frequent hazards created by vehicles passing on a hill or curve, or entering the highway. The guide line on some sections has been painted continuous for the entire length; on others a continuous line on curves and hills with an intermittent or no line between. The guide lines on U. S. Route No. 1 in New Hampshire have been painted double the width on other sections. White and orange are the principal colors used for guide line painting.

Due to the studies being confined to fact-finding in connection with the maintenance costs, no endeavor is being made to determine why the States have different standards and methods.

The States are performing annual maintenance to keep the selected sections in the same condition for the duration of the studies. Field inspection is being made annually to determine any variation in the degree of maintenance.

The States are furnishing reports of annual cost of general highway maintenance on each section. The forms used for reporting the cost data are shown on pages 300-301 and 304-305 of the report of the Committee on Maintenance Costs.

It is noted that maintenance costs are shown for six main divisions and subdivisions, also other data essential for an analysis of this kind.

Maintenance costs are being analyzed for each State separately and then the maintenance costs for similar items for the various States will be compared.

In order to make a comparison of the maintenance costs reported on B. P. R. Maintenance Cost Form M-1 it is necessary to select units of measurements that can be used for each section which are as follows:

For surfacing:
- Cost per thousand square yards.
- Cost per lane mile.

For shoulders:
- Cost per thousand square yards.
- Cost per shoulder mile.

For drainage:
- Cost per hundred linear feet of ditches.
- Cost per hundred linear feet of drains.
- Cost per culvert or per hundred linear feet for culvert cleaning.

For structure repairs:
- Cost per bridge.
- Cost per culvert.
- Cost per hundred linear feet of guard rail.

For roadside cleaning:
- Cost per acre.

For traffic service:
- Cost per hundred highway markers.
- Cost per hundred linear feet of guide line painting.
- Cost per section mile for snow removal.
- Cost per section mile for ice control.
- Cost per hundred feet of snow fence.

There are other items for which proper units of measurement have been selected.

Due to the effect of traffic on the surfacing and the maintenance costs incurred being greater than on any other part of the highway, it is the most important item to be studied. In order that various and many summaries may be made, the maintenance costs for concrete surfacing are set up in the manner shown on Table 5 as an example.

It is noted that the maintenance costs for each year will be analyzed and summaries made to show the total and average cost of maintenance per thousand square yards and per lane mile for one, two, three, four, and five years. A tabulation of this kind for each section will permit the summarization of the maintenance costs for each type of surface for the first, second, third, fourth, and fifth year; also for one, two, three, four, and five years.

The next step in the analysis is to determine the effect of traffic on the sur-

TABLE 5

MAINTENANCE COST STUDIES

Inventory							Maintenance cost									
								Cost of patching			Cost of joint and crack filling			Cost of all items		
Section	Town location	Year built	Length of section miles	Width of pavement lin. feet	Area of pavement sq. yds.	Traffic lane miles	For year ending	Total	Per 1,000 sq.yds.	Per lane mile	Total	Per 1,000 sq.yds.	Per lane mile	Total	Per 1,000 sq.yds.	Per lane mile
C1	Gilsum	1930	2.936	20	34,444	5.872	1-31-36	$ 0.00	.....	.....	$25.14	$ .73	$4.28	$ 25.14	$ .73	$4.28
							1-31-37	0.00	.....	.....	45.45	1.31	7.74	45.45	1.31	7.74
						Total for two years	.....	.....	.....	.....	70.59	2.04	12.02	70.59	2.04	12.02
						Annual average for two years	.....	.....	.....	.....	35.30	1.02	6.01	35.30	1.02	6.01
							1-31-38	38.44	$1.12	$6.55	123.97	3.60	21.11	161.41	4.72	27.66
						Total for three years	.....	38.44	1.12	6.55	194.56	5.64	33.13	233.00	6.76	39.68
						Annual average for three years	.....	12.81	.37	2.18	64.85	1.88	11.04	77.66	2.26	13.22
							1-31-39	30.75	.89	5.24	0.00	.....	.....	30.75	.89	5.24
						Total for four years	.....	69.19	2.01	11.79	194.56	5.64	33.13	263.75	7.65	44.92
						Annual average for four years	.....	17.30	.50	2.95	48.64	1.41	8.28	65.94	1.91	11.23
							1-31-40	.....	.....	.....	.....	.....	.....	.....	.....	.....
						Total for five years	.....	.....	.....	.....	.....	.....	.....	.....	....	.....
						Annual average for five years	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
C2	Dublin	1927	1.752	18	18,510											
		1928	1.666	18	17,589											
		1929-30	3.561	20	41,777											
		1934	2.377	20	27,895											
			9.356		105,771	18.712	1-31-36	97.04	.92	5.18	95.16	.90	5.09	192.20	1.82	10.27
							1-31-37	73.98	.70	3.96	199.29	1.88	10.65	273.27	2.58	14.61
						Total for two years	.....	171.02	1.62	9.14	294.45	2.78	15.74	465.47	4.40	24.88
						Annual average for two years	.....	85.51	.81	4.57	147.23	1.39	7.87	232.74	2.20	12.44
							1-31-38	182.16	1.72	9.73	374.73	3.54	20 02	556.89	5.26	29.76
						Total for three years	.....	353.18	3.34	18.87	669.18	6.32	35.76	1022.36	9.66	54.64
						Annual average for three years	.....	117.73	1.11	6.29	223.06	2.11	11.92	340.79	3.22	18.21
							1-31-39	121.51	1.15	6.49	104.10	.98	5.56	225.61	2.13	12.05
						Total for four years	.....	474.69	4.49	25.37	773.28	7.30	41.32	1247.97	11.79	66.69
						Annual average for four years	.....	118.67	1.12	6.34	193.32	1.83	10.33	311.99	2.95	16.67
							1-31-40	.....	.....	.....	.....	.....	.....	.....	.....	.....
						Total for five years	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
							.....	.....	.....	.....	.....	.....	.....	.....	....	.....

NOTE: The above amounts are used for illustrative purposes only.

facing. For this purpose the weights of the various types of vehicles traversing the highways in New Hampshire and Rhode Island have been furnished by the State-wide Highway Planning Surveys and similar information will be furnished for Connecticut as soon as the field work by the surveys is completed.

Since this is a five-year study which will not be completed for another year, no results or conclusions are formulated at this time.

DEPARTMENT OF TRAFFIC

ARNOLD VEY, *Chairman*

LEGIBILITY DISTANCES OF HIGHWAY DESTINATION SIGNS IN RELATION TO LETTER HEIGHT, LETTER WIDTH, AND REFLECTORIZATION

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AND

ROBERT S. HOLMES

Pennsylvania Turnpike Commission

SYNOPSIS

A previous paper by one of the authors analyzed sign efficiency into (1) pure legibility (long time), (2) glance legibility (short glance), (3) target value and (4) priority value, the latter two factors dealing with attention getting characteristics. This report deals with an extensive study of the first factor, pure legibility, of standard series B (narrow letters) and series D (wide letters), black on white destination signs. Letter heights from 6 to 24 inches were used.

A total of about 400 people acted as observers and made about 4600 individual observations. Charts show maximum legibility distance in relation to height and width of letter, reflectorization and illumination (day vs. night). The determination for each size, width and type of letter depended on a group of from 50 to 276 observations.

The wider letters showed greater effectiveness than the narrower ones, but both widths lost approximately 15 per cent of their legibility distance under night conditions (floodlighted). Rough practical norms for day conditions and "normal" vision gave 50 ft. per in. of letter height for the wide and 33 ft. per in. for the narrow letters. It is suggested that these values should be reduced in practice to take account of the poor vision of many drivers known to be on the highways.

The results also indicated the advantage of floodlighted signs at any location where a lighted background is encountered. Reflectors reduced the daytime legibility of the letters only in the largest sizes, and the reflectorized letters were as effective as the floodlighted ones at night against a dark background.

It is suggested that glance legibility tests will probably reduce the legibility distances somewhat, and that a correction factor may be developed, to correspond more closely to brief glimpses of signs often necessitated by heavy traffic.

If the results are applied to give the motorists 10 seconds warning at a speed of 50 miles per hour, *at least* a 10 inch series D or a 12 inch series B letter is needed for night conditions.

A number of traffic officials and others interested in traffic engineering have come to the opinion that methods of improving the effectiveness of certain types of highway signs should be studied. Varying opinions are found as to what characteristics make a highway sign seen most easily and the standard manual on Uniform Traffic Control Devices represents an attempt to standardize and crystallize the best opinion on this subject.

There is, however, still a wide field in which lack of agreement is characteristic. Specifically; is it more effective to use a 6-in. high series D letter or a 10-in. series B? The latter is relatively narrower and hence can be fitted in where the wider letter could not. This problem is especially pertinent for long names on destination signs. Another specific question is whether or not floodlighted signs are more efficient and can be read farther than reflectorized signs, again thinking of the destination sign. A problem which

arises on newer highways due to an increase of design speed is that of the oversized sign which is recommended by the Standard Manual when conditions warrant. What size of letter should be indicated to give a certain amount of warning or legibility distance? Can the legibility of smaller letters be multiplied up on a proportional basis to obtain legibility distances for large signs? And what distances can be used for letters of various proportions?

A method for the analysis of highway sign efficiency which will answer these questions has been worked out and demonstrated to be practical in a previous paper.[1] Four basic factors of sign efficiency were derived as follows: legibility, glance legibility, target value, and priority value. This analysis followed in rough fashion other analyses that had been made in the field of advertising, but with modifications to meet the highway problem.

Pure legibility was used to indicate the maximum distance at which sign copy could be read under optimum conditions, *i. e.,* with no distraction and unlimited time. Glance legibility indicated the distance at which a sign could be read under quick glance conditions, such as those where a driver must attend to other cars and has just a fleeting glimpse of the sign. Target value indicated that characteristic of a sign which makes it stand out as different from other signs and objects. Priority value was applied to the characteristic which causes one sign to be seen first from among a number of other very similar or identical signs.

The present paper reports a more extensive study of the first factor, *i. e.,* pure legibility distance, of standard black on white destination signs, relating two-letter widths and letter height to legibility distance under day and night conditions. It is hoped that studies of other factors may be carried out later by ourselves or others.

[1] Forbes, T. W., A Method for Analysis of the Effectiveness of Highway Signs. *Journal, Applied Psychology,* 1939.

Method

A large number of observations were obtained using standard destination signs under both night and day conditions with reflectorized and unreflectorized letters. Figure 1 shows how eight different signs

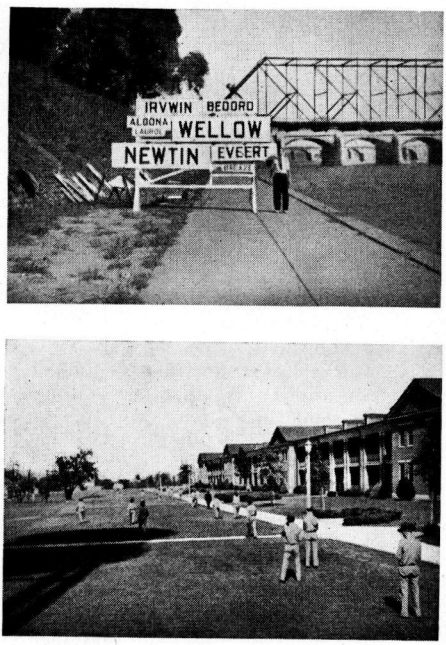


Figure 1

Upper—Series D Signs Mounted for Day Test.

Lower—Observations in Progress, Illustrating Scatter of Different People on the Same Sign.

were mounted at one time. The observers were first stationed at a distance of about 1400 feet where they were unable to read any of the signs. They then approached the signs and recorded the letters seen and the distance at which they were first able to read them. Distances were chalked on the pavement at 20-ft. intervals. By this modification of the previous technique it was possible to utilize

the observers in groups of 30 or 35 and thus expedite the amassing of a large number of observations. The observers were on foot and three supervisors checked to insure proper recording.

Test Signs

Six-letter words were painted in black on white boards. Words were chosen from the vicinity of the Pennsylvania Turnpike to add realism to the test and each six-letter word included one misspelling in order to guard against guessing by the observers. This precaution proved important since some 400 faulty observations (in which the misspelling was not recorded) had to be thrown out from the approximately 5000 total obtained. Five variations of each test word were thus obtained. This allowed the use of a different series of test words for each group of observers. In this fashion it was possible to present to each group a new test in spite of discussion of the tests by members of the same organization.

Six letter heights were used in the plain letter for each of two widths (series D and series B). Three letter heights were provided in the reflectorized series D letter. The reflectors used were of glass and for the 6-in. letter were 0.5 in. in diameter; for the 12 and 18-in. letter they were 0.7 in. in diameter.

Letter Proportion and Spacing

Letter dimensions and proportions were obtained from working drawings for series B and series D letters as listed in the Manual for Uniform Traffic Control Devices.[2] For convenience these dimensions are shown in Table 1.

Since spacing is known to be of importance it was desirable that the spacing used should be definitely specified. In determining the spacing several alternatives from usual practice appeared. Constant spacing was discarded because it gives a poor balance and therefore poor appearance. A varied spacing was obtained which had been derived for practical use [3] and which was satisfactory as to appearance, but involved some 50 combinations of letters and spacing.

In order to make this system more practical, it was necessary to reduce the spacing to a simpler basis, which could be used in the sign shop. This was done by approximating the spacing and by expressing all spacings in terms of *adjacent strokes*, rather than *adjacent letters*. Thus three spacings resulted as follows: spacing between (1) parallel strokes, (2) a slanting and a vertical stroke, and (3) strokes slanting toward each other.

On this basis it was found practical and fairly easy to lay out lettering for the sign painter and the results were always predictable. This spacing attempts to equalize somewhat the area between letters due to different letter shapes. Table 2 gives the detail of the three spacings together with illustrations of their applications.

For night observations the upper half of the signboard containing unreflectorized signs was floodlighted. This was done by mounting seven 50-watt goosenecked lamps to shine directly on the test signs. Power was obtained from the batteries of two cars in order to obtain easy mobility and to be able to operate where power was not available from city lines. A switch was supplied to cut off the floodlights when observing reflectorized signs under headlights alone. Illumination intensity as measured by a Weston light meter ranged from 5 to 15 footcandles. A reflection factor of about 0.3 was indicated after the signs had become somewhat dirty from use and handling, so that an average brightness of about 3 footlamberts was estimated to be not far wrong.

[2] Obtained from the Public Roads Administration, Washington, D. C.

[3] By the Signal Service Corporation.

TABLE 1

PENNSYLVANIA TURNPIKE COMMISSION

Dimensions for Stencil Letters and Numbers for Large Signs

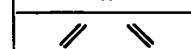
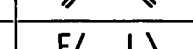
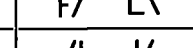
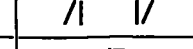
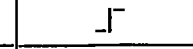
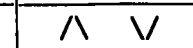
Height inches	Stroke inches	WIDTH-INCHES											
		I	1	EF LT	BCDGHN OPQRSUZ 23567890	4	J	K	V	M	AY	W	X
						SERIES "B"							
4	1/2	1/2	5/8	1 1/2	1 11/16	1 15/16	1 19/32	1 3/4	1 27/32	1 15/16	2 1/8	2 3/16	1 27/32
6	3/4	3/4	15/16	2 1/4	2 1/2	2 7/8	2 3/8	2 5/8	2 3/4	2 7/8	3 1/8	3 1/4	2 3/4
8	1	1	1 1/4	3	3 3/8	3 7/8	3 3/16	3 1/2	3 11/16	3 7/8	4 1/4	4 3/8	3 11/16
10	1 1/4	1 1/4	1 9/16	3 3/4	4 3/16	4 13/16	4	4 3/8	4 5/8	4 13/16	5 1/4	5 7/16	4 5/8
12	1 1/2	1 1/2	1 7/8	4 1/2	5	5 7/8	4 3/4	5 1/4	5 1/2	5 7/8	6 1/4	6 1/2	5 1/2
18	2 1/4	2 1/4	2 7/8	6 3/4	7 1/2	8 3/4	7 1/8	7 7/8	8 1/4	8 3/4	9 3/8	9 3/4	8 1/4
24	3	3	3 3/4	9	10	11 1/2	9 1/2	10 1/2	11	11 1/2	12 1/2	13	11
						SERIES "D"							
4	11/16	11/16	7/8	2 7/16	2 11/16	2 15/16	2 1/2	2 3/4	3	3 3/32	3 11/32	3 1/2	2 11/16
6	1	1	1 1/4	3 5/8	4	4 3/8	3 3/4	4 1/8	4 1/2	4 5/8	5	5 1/4	4
8	1 3/8	1 3/8	1 3/4	4 7/8	5 3/8	5 7/8	5	5 1/2	6	6 5/16	6 11/16	7	5 3/8
10	1 11/16	1 11/16	2 3/16	6 1/16	6 11/16	7 5/16	6 1/4	6 7/8	7 1/2	7 3/4	8 3/8	8 3/4	6 11/16
12	2	2	2 1/2	7 1/4	8	8 3/4	7 1/2	8 1/4	9	9 1/4	10	10 1/2	8
18	3	3	3 3/4	10 7/8	12	13 1/8	11 1/4	12 3/8	13 1/2	13 7/8	15	15 3/4	12
24	4	4	5	14 1/2	16	17 1/2	15	16 1/2	18	18 1/2	20	21	16

For reflectorized signs the headlights of a car were trained upon the three signs on the lower half of the signboard. The car proceeded abreast of the group of observers so that the light was always roughly at the point of observation. For the majority of observations a 1938 Chrysler was used with newly replaced headlight bulbs, giving approximately 50,000 beam candles. The hot spot was aimed directly ahead and depressed so as to strike 32 in. above the ground 200 ft. ahead of the vehicle when loaded. (Top of hot spot about 1.4 deg. below the horizontal.) Observers stood on the rear bumper so that a loaded condition was obtained. The signs were directly in front of the car and therefore illumination should have been somewhat better than ordinarily obtained on the shoulder. However, the hot spot did not fall directly on the signs at any point, a condition similar to that under which most signs are seen on highways.

The mounting of the signs was such that the highest portion of the reflectorized signs was between five and six feet above the road, and the lowest approximately three feet in an attempt to obtain mounting heights somewhat similar to those ordinarily found and indicated as standard by the Manual.

TABLE 2

SPACING SYSTEM FOR STANDARD CODE LETTERS

Code letter	Spacing between	Illustration	
X	Parallel strokes.		OBN
			AV
	F, L, J and parallel stroke.		LVJ FA
Y	Sloping and vertical stroke.		VBAKD
	T, E, F, L and vertical stroke.		EFB LT
Z	Opposite sloping strokes.		VY KA
	T, E, F, L and opposite sloping stroke.		TVLAJ

NOTE: Spacing is measured between two nearest points of adjacent letters.

Letter height	Spacing								
	Series "B"			Series "C"			Series "D"		
	X	Y	Z	X	Y	Z	X	Y	Z
6	7/8	5/8	1/2	1 1/8	7/8	5/8	1 5/16	1 1/16	3/4
8	1 1/8	7/8	5/8	1 1/2	1 3/16	13/16	1 13/16	1 7/16	1
10	1 3/8	1 1/8	3/4	1 13/16	1 7/16	1	2 3/16	1 11/16	1 3/16
12	1 11/16	1 5/16	15/16	2 1/4	1 3/4	1 1/4	2 11/16	2 1/16	1 1/2
18	2 1/2	2	1 3/8	3 3/8	2 5/8	1 7/8	4	3 1/8	2 1/4
24	3 3/8	2 5/8	1 7/8	4 1/2	3 1/2	2 1/2	5 3/8	4 1/8	3

Personnel

Some difficulty was experienced in obtaining sufficient personnel and for that reason the total number of observers is smaller than had been planned originally. It was found possible to hold each group of observers about one-half hour, which was the time required for a complete set of observations, on eight series D and five series B signs. The observers consisted of engineering office personnel, members of the national guard, and enrollees of an army medical school. The ages of the last two groups tended toward the lower end of the scale, and both groups would be expected to have better vision than some drivers on the road. This factor was accounted for later.

Groups were scheduled in a fashion as convenient as possible for the personnel.

Results

A total of 4,623 observations was obtained from 412 different persons divided between day and night groups. Each plotted point in the curves is based on from 50 to 276 observations as shown on each distribution.

The various observers were able to read a given test sign at widely different distances, as would be expected. Thus, for a given letter height (see Fig. 2 as an example) the legibility distances showed ranges of from 300 to roughly 700 ft. It was therefore necessary to present the results in the form of a frequency distribution.

For our purposes, the most reliable measure obtainable from such distributions is probably the median, or middle case, and this value has therefore been indicated and a median trend line shown. Similarly the 80 percentile values, *i. e.*, the distance at which 80 percent of the group were able to read the sign, has been determined and its trend line indicated. These 80 percentile values are probably of most interest for the determination of practical sign legibility distances. Furthermore, 95 percentile points have been indicated in order to show the distances for those with the poorest vision. These 95 percentile points are not of as great statistical reliability since a much larger sample would be required to determine accurately the points at the extremes of the distribution.

Figures 2 to 6 show the results for the wider, or series D letter, while Figures 7

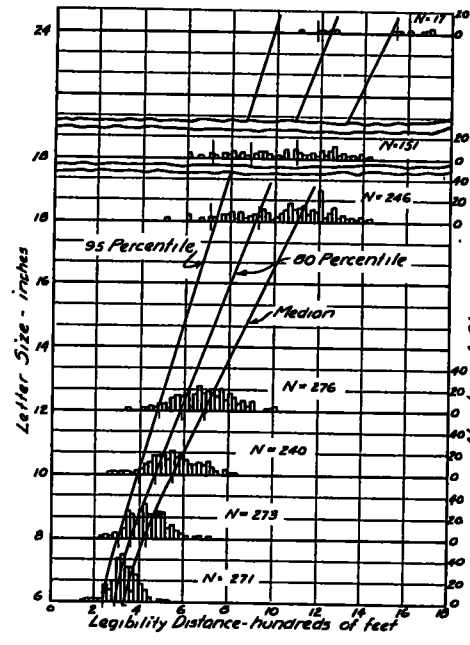


Figure 2. Daylight Legibility, Black on White, Series D Letters, Unreflectorized

and 8 show those for the narrower, or series B letter. Each figure represents observations made by the same group of people on the six-letter test words for letter sizes of 6, 8, 10, 12, and 18 inches, as indicated. One very small group of observations is shown in Figure 2 for 24-in. letters. It was unfortunately impossible to obtain more observations with this size sign.

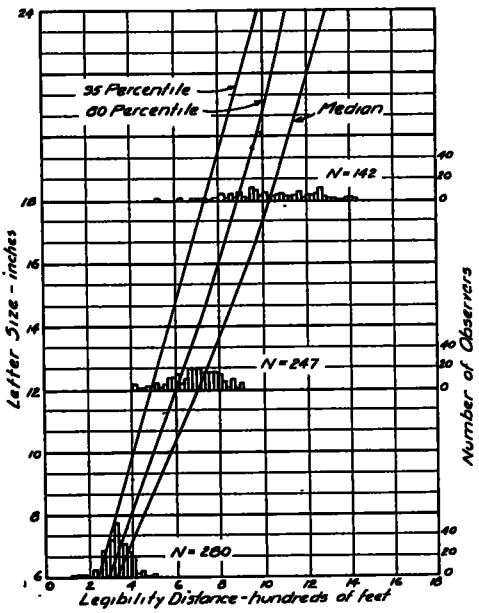


Figure 3. Daylight Legibility, Black on White, Series D Letters, Reflectorized

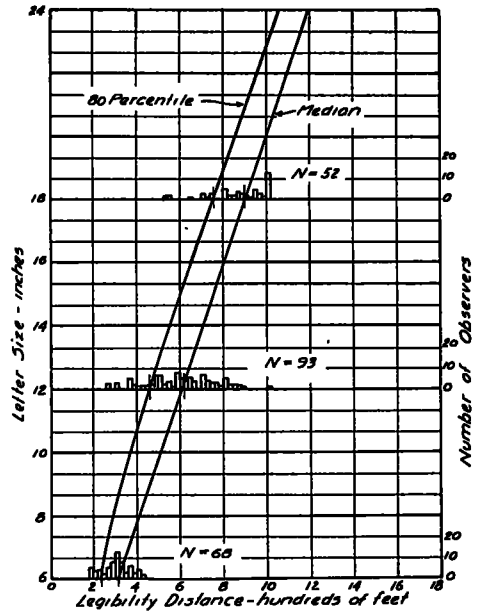


Figure 5. Night Legibility, Headlights, Black on White, Series D, Reflectorized. Dark Field, Scattered Street Lights.

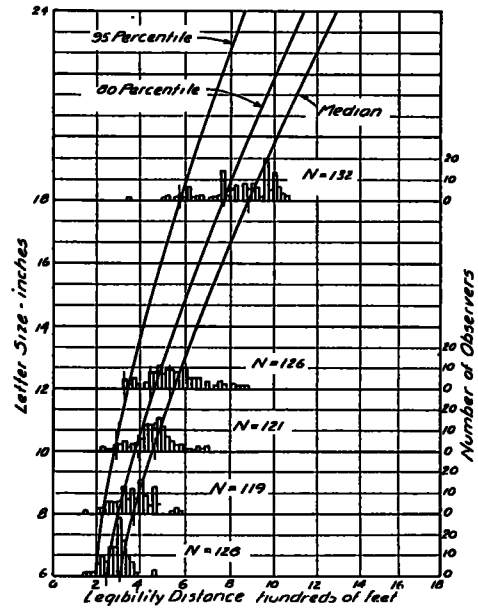


Figure 4. Night Legibility, Floodlights, Black on White, Series D Letters, Unreflectorized. Illumination 5-15 Foot Candles, Average Brightness 3 Foot Lamberts.

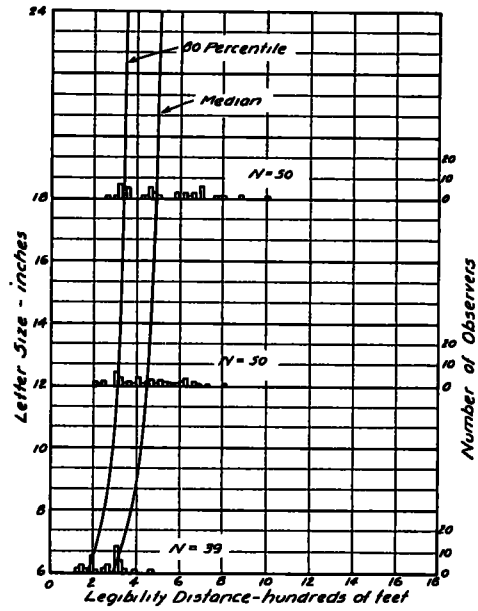


Figure 6. Night Legibility, Headlights, Black on White, Series D Letters, Reflectorized. Lighted Field, 3 Foot Lamberts. Scattered Street Lights.

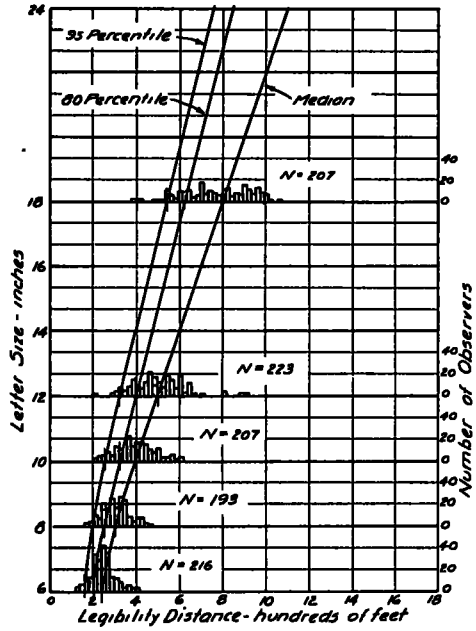


Figure 7. Daylight Legibility, Black on White, Series D Letters, Unreflectorized

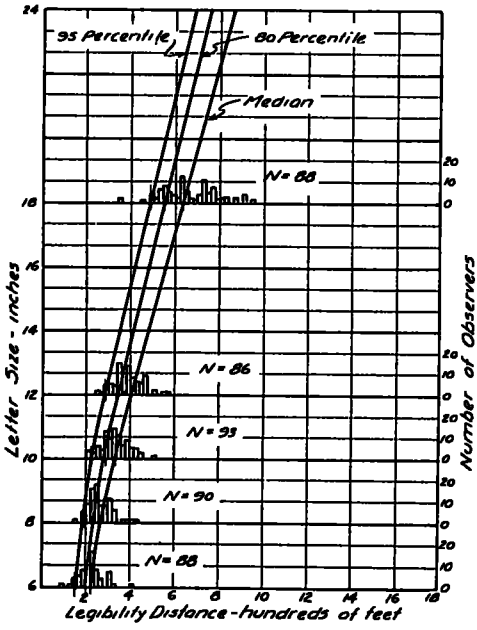


Figure 8. Night Legibility, Floodlights, Black on White, Series B Letters. Illumination 5-15 Foot Candles, Average Brightness 3 Foot Lamberts.

Trend Indices

It will be seen that the medians fall in a very consistent fashion and that the trend varies slightly from a straight line. The 80 percentile trends are generally similar to those for the medians, but show a shorter legibility distance to the extent of from 50 to about 200 feet. The 95 percentiles show a greater variation from the median than the 80 percentile, as would naturally be the case, and also a poorer conformity of the plotted points through the trend line, indicating poorer reliability for this measure.

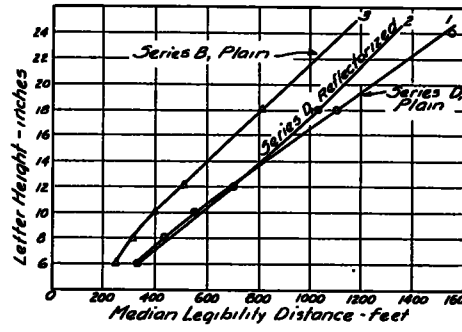


Figure 9. Effect of Width and Reflectorization, Daylight Median Values. Series D, Wider Letters. Series B, Narrower Letters.

Therefore, for purposes of comparison of letters under different conditions, medians will be used, whereas practical legibility distances will be based on 80 percentile figures. The range of vision represented will be shown later to give a further reason for choice of the 80 percentile as a base for practical values.

Day Legibility

The results for the different letters under day legibility conditions are summarized in Figure 9. As shown by the medians, the narrow letters (series B) gave from 100-300 feet less legibility distance than the wider letters for the range of letter sizes compared. The reflectorized letters showed practically no differ-

ence in legibility from the plain or unreflectorized letters in the smaller letter sizes.

Night Legibility

The night observations again show a similar relationship between the wide and narrow letters as indicated by the medians in Figure 10. Here again it will be seen that the narrow letters gave from 80 to 250 feet less legibility distance, when unreflectorized letters are compared.

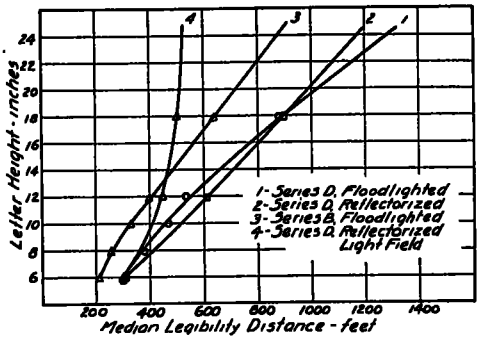


Figure 10. Effect of Width and Reflectorization at Night. Median Values

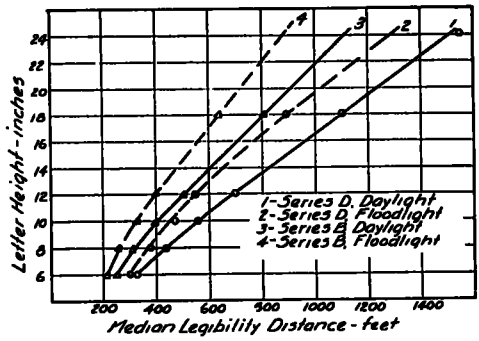


Figure 11. Comparison of Night and Day Legibility. Median Values

Day versus Night Legibility

When the legibility distances for the wide letters are compared on a day and night basis, it appears, as shown in Figure 11, that the floodlighted letters gave from 25 to 200 feet less legibility (8 to 20 per cent). Similarly for the narrower, or series B letters, the loss under floodlighting at night was from 30 to 150 feet of legibility distance (12 to 20 per cent). It will be noted that the loss was proportionally greater for the higher letters and longer distances, a result which is probably explainable as due to low effective brightness of the signs at long distances at night.

Effect of Reflectorization

As previously noted, it was possible to reflectorize only series D letters, and therefore conclusions must be limited to results from the wide letters. With this reservation, however, the effect of the reflectors in the black stroke of the letter was negligible under day conditions up to the 12-in. size (see Figure 9). For the larger letters (18-in.) there appears to have been some loss in legibility distance.[4] This is understandable since reduced contrast would be more deleterious at a long distance where the effective brightness of the signs is very much reduced.

The night legibility of the reflectorized letters, when viewed under headlights and in a dark field, was approximately the same as for our floodlighted letters for the same height and width (see Figure 10). The condition referred to as "dark field" includes some scattered street lights of low wattage but no field of light close to the sign.

On the other hand, the same reflectorized letters under the same headlights lost from 30 to 50 per cent of their legibility distance when viewed against a low-intensity light field directly above them. The light field consisted of the lighted signs of approximately three foot-

[4] This was not due to the smaller number of cases as shown by comparison with the insert in Figure 2 which represents the smaller group on plain letters.

lamberts brightness. The reduction occurred in the two larger sizes as shown in Figure 10. Apparently any increase in size above 12 inches under these conditions resulted in very little increase of effective distance. This points out the need for greater intensity of reflected light if the signs are to be viewed against any lighted background, providing the irradiation from the greater intensity does not offset its advantage. Conversely, reflectorized signs would seem to be most effective in locations where a dark background is to be expected a large proportion of the time.

Practical Distance Values

In order to determine from our observations the practical distance at which signs of a given letter height and width are legible to the driving public, it is necessary to make some estimate of the vision of our observers as related to that found on the highway. Such an estimate was attempted by giving the ordinary Snellen eye test (chart) to two groups of observers. Their records for the 6-in. test words were then compared with their record on the Snellen test. Figure 12 shows a comparison of the Snellen and the sign observations and indicates the method of deriving the visual scale represented by the 6-in. letters in daylight.

In order to determine a point corresponding to 20/20 vision, the median of the 20/20 group (eye test) was assumed to correspond to the score of this median person on the 6-in. letters. Other acuities were then figured on a proportional basis and the scale plotted as shown. Its approximate correctness is indicated by the rough correspondence of the range of eye test scores and of computed vision for the two small groups. Unfortunately some individuals were transferred from groups 4 and 5 before the eye test was given, so that group 1 gives the more reliable index.

By this procedure, it was indicated that our 80 percentile point represented approximately 20/20 vision, and our 95 percentile approximately 20/30 vision. In other words, the vision of our observers was better than average in eye test terms.

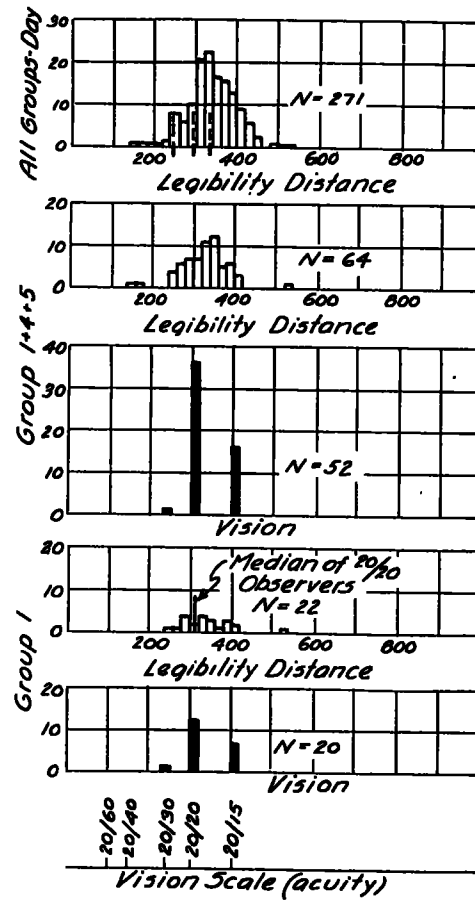


Figure 12. Eye Tests and Legibility Distance Snellen Tests on Two Subgroups of Observers Related to Their 6-Inch Legibility Distance. Visual Scale Derived from This Comparison Indicates Range of Vision Represented in Total Group of Day Observers.

Median of 20-20 Eye Test Group Equated to Corresponding 6-Inch Legibility Distance. Visual Scale Computed Proportionally from This Point.

This was to be expected, since our army and national guard observers were a selected group and since the engineering

office personnel undoubtedly had better visual correction than many whose job does not call for use of the eyes on close work. On the basis of these results, our 80 percentile distances may be taken as representing 20/20 vision and should be used rather than the medians in order to be conservative.

However, it should be remembered that 20/40 vision (50 per cent vision) is used to qualify drivers in a number of states and therefore a certain proportion of drivers of this type is to be expected on any highway. Distances corresponding to 20/40 vision can be determined by

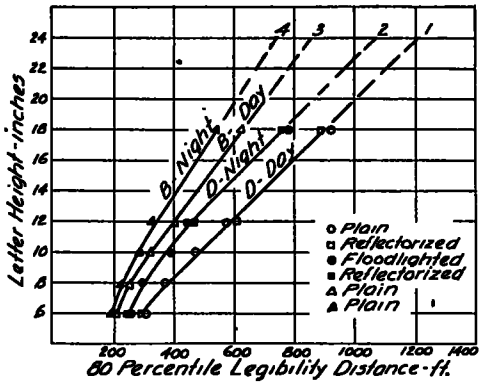


Figure 13. Practical Legibility Values Corresponding to 20/20 Vision, 80 Percentile Figures.

taking 50 per cent of the distance shown on our chart in Figure 13.

In deriving the practical values shown in this figure the plain and reflectorized values for the wide or series D letter under day conditions have been thrown together. Similarly, the floodlighted and reflectorized values for night observations have been lumped in order to give a single approximate line of relationship. (Although the medians indicate that the difference between plain and reflectorized letters is a real one, nevertheless, for practical purposes the 80 percentiles may be considered roughly the same, as will be seen from the scatter of the points around the line.) Curves for the series B 80 percentiles are also included. It will be remembered that the 80 percentiles are from 50 to 200 feet shorter than the median values.

Comparison by Letter Width

One of the original questions which was posed in the introductory paragraphs may now be answered from Figure 13. The values shown indicate that under day conditions, the narrower 10-in. series B is slightly more effective in terms of legibility than the 6-in. series D, and the 8-in. series B is approximately 80 per cent as effective as the 6-in. series D letters. A similar relationship is found for night conditions.

On the other hand, during the day an 18-in. series B letter shows approximately the same maximum legibility as a 12-in. series D, while under night conditions the 18-in. series B is about 22 per cent more effective than the 12-in. series D. Or, stated in another way, at night the 12-in. series D and the 15-in. series B are approximately equal. Any other comparisons may be made in similar fashion as desired.

Practical Importance of Nonlinearity

As noted above, for practical purposes the relationship of letter height and width to legibility distance may be stated in terms of approximate straight lines. Actually, a curvilinear characteristic is noticeable between 6 and 10-in. values. The variation represented by the points on the curve, when translated into terms of visual angle, correspond to slight fractions of one minute of arc. In fact, as shown in Figure 14, the variations between the night and day values (series D) falls between a theoretical constant minimum visual angle of 1 min. and a constant angle of 1.5 min. Variations in acuity have been demonstrated from 0.5 to 7.0 min. due to a difference in brightness which would correspond to part of

our range.[5] Any such variation must be the resultant of complex reactions of the eye and is beyond the scope of this paper, except that it indicates our variations to be reasonable.

The practical importance of a curved characteristic relationship between letter size and legibility distance is obvious if an attempt is made to project the relationship found with small test letters to make it apply to larger sized letters. If brightness or other relationships result

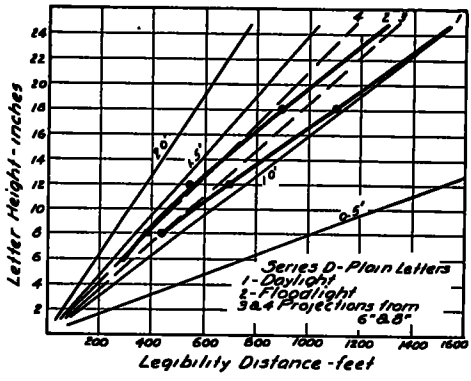


Figure 14. Error from Projection. Dotted Lines Indicate Projected Values from Small Letters Alone on the Assumption of a Linear Relationship.

Lines Marked 0.5', 1.0', 1.5' and 2.0' Represent Theoretical Values Assuming the Indicated Minimum Constant Visual Angle.

in a curvilinear relationship, it becomes much more difficult to extrapolate the curve for larger signs and makes it necessary to know the characteristic curvature fairly definitely.

It will be noted that if only the smaller sized letters had been employed the relationship might easily have been interpreted as linear. The error which would have occurred from the extrapolation of such a linear trend line is shown in Figure 14.

However, the use of smaller letters for test purposes is much easier and more convenient and would be possible if the curved characteristic is known.

DISCUSSION

Obviously many other factors which were not included in our situation will affect the practical distance at which signs are read under traffic conditions on the highway. A few other points also deserve brief consideration.

Vision of Driver

We have considered the vision of our observers as related to one of the usual eye test norms and have thereby corrected our results to correspond to 20/20 vision. There remains the question of what proportion of the drivers on the highway actually have average vision and what proportion have various degrees of poorer vision. Reliable data on this subject are scarce, but one report[6] shows out of a total of 35,040 drivers tested at automobile shows, safety conventions, etc., that about 7.3 per cent had a vision corresponding to about 20/33, and 24.2 per cent had vision corresponding to 20/25 (60 and 80 per cent vision respectively). If all those had been ruled off the road whose vision was poorer than 20/40, or 50 per cent vision, the figures would be reduced by about 1 per cent. If we disregard the question of whether or not those tested at such gatherings represent a true sample of drivers on the highway, these figures would suggest that a vision figure of not better than 20/30 should probably be used. As previously noted, this would correspond roughly to our 95 percentile points.

[5] Ferree, C. E., & Rand, G. "The effect of intensity of illumination on the acuity of the normal eye and eyes slightly defective as to refraction." *American Journal of Psychology*, Vol. 34, p. 244, 1923.

[6] Allgaier, E., American Automobile Association, "Interrelationships of Driver Test Scores," Research Report No. 11, 1939.

However, the choice of a given value seems inadvisable due to the many aspects involved and we have therefore preferred to merely indicate our 80 percentile values as corresponding to 20/20 vision ("average" or "normal" vision).

An application of the findings on sign legibility may be of importance in relation to the question of eyesight standards for drivers. Other organizations are working on this question.

Glance Legibility

Other factors tending to modify the legibility values obtained are those of familiarity, comfortable vision, and glance legibility. Familiarity with the sign can be shown to increase its effective reading distance, while the driver who does not strain his eyes, but waits for a comfortable distance (of an unfamiliar sign), would read at a shorter distance than the maximum value which we have obtained. However, these two factors are somewhat imponderable and cannot be easily included in a reproducible measure.

The third factor, glance legibility, is probably the most important of the three, since it is involved where distractions arise from other traffic and where blocking out by other cars allows the driver only a short duration glance at the signs. The preliminary study showed that this factor is measurable and that it probably results in a shortening of the practical legibility distance. It is to be hoped, therefore, that a further study, perhaps by other investigators, may be carried out in which a correction or multiplier factor may be derived for expressing glance legibility as a proportion of the pure legibility distances which we have measured. Such a factor would be convenient since the long time "pure legibility" values are easier to measure than the short time values.

Relation to Glare

The light field which caused a reduction of from 30 to 50 per cent in the legibility distance accruing to the larger reflectorized letters should be carefully distinguished from the intensities ordinarily thought of as glare in connection with headlighting. As previously noted, our intensities of illumination were from 5 to 15 footcandles with a reflection of approximately $\frac{1}{3}$ of this amount. Therefore, the average brightness toward the eye of the floodlighted signboard which constituted our light field was approximately three footlamberts as compared to 2000 candlepower from a point source, which has been shown by others to cause a 50 per cent reduction of visibility of a gray figure under headlights.[7] The latter visibility, however, was from 200 to 300 feet under bright headlights and the illumination of the object was therefore many times higher than the intensity of the light from the reflectors in our signs at from 500 to 900 feet. On this basis, the blanking effect of the relatively low brightness which was found in our study is understandable.

The relatively great loss in effectiveness due to the light field calls attention to the value of floodlighted signs in any location where a light field of any intensity or extent is met. However, on an open highway where dark conditions prevail for a great proportion of the time, the values for the reflectors would seem to be equally good. Although headlights present a high intensity, the short time during which they are ordinarily facing the driver would mean that their effect on reflectorized sign legibility would be of short duration except under dense traffic conditions.

[7] Reid, K. M., "Seeing on the Highway." *Proceedings*, Highway Research Board, Vol. 17, p. 420, 1937.

Relation to Fifty Foot Rule

A rule of thumb which is in use by some of those in the practical sign field, gives 50 ft. of legibility distance per inch of letter height. It will be noted that our results show a good correspondence to this rule for the wider series D letters under day conditions, but roughly 15 per cent shorter distance than this for night conditions. The series B letters, on the other hand, result in more nearly 33 ft. to the inch under day conditions, and again with a 15 per cent loss at night.

Speed and Warning Distance

On the basis of the pure legibility values, however, it is interesting to note the size of letter indicated if a certain warning time and speed are assumed. Assuming a value of 10 sec. warning, which has been suggested by some experts, and a conservative speed of 50 miles an hour, a distance of 750 ft., or an 18-in. series D sign (night values) is indicated. On the same assumption, but with the sign placed 400 ft. ahead of the hazard as recommended by the Manual for Control Devices, a 10-in. series D, or a 12-in. series B sign is indicated for night conditions. The curves in Figure 13 also point out dramatically that the night condition is the critical one from the point of view of sign legibility.

SUMMARY

1. A study of pure legibility distances has been made using standard black on white destination signs with 6 to 18-in. letters of two widths (series B and series D). Total observations numbered approximately 4,600 and the total number of observers 412. Each plotted point on trend curves was derived from distributions of from 50 to 276 observations. A technique was used which eliminated guessing by observers. Test conditions included day and night, reflectorized and unreflectorized signs, floodlights and headlights.

2. Specified spacings were developed by a simplification of a more complex spacing. Spacings were specified in terms of adjacent strokes rather than letter combinations. This system proved practical for laying out signs.

3. Since median values are most reliable, they were used as an index for comparing trends and relative values. However, our 80 percentile values were shown to correspond roughly to average, or 20/20 vision, and were therefore used for deriving practical distance values.

4. Curves were presented which allowed a comparison of the relative efficiency of the different letters used in terms of pure legibility, e.g., at night a 12-in. series D letter gave approximately the same legibility distance as the 15-in. series B.

5. Reflectors reduced the daytime legibility of the smaller letters very little but showed some effect on the 18-in. letters.

6. The night legibility of the reflectorized signs against a dark background was approximately the same as for the floodlighted signs. However, when the floodlighted signs were left illuminated to a brightness of about three footlamberts, a reduction of 30 to 50 per cent in the legibility distance of the larger reflectorized letters occurred. This brightness corresponds to the less bright display signs and is very much lower than the brightness of oncoming headlights. The value of floodlighted signs where constant light backgrounds occur is indicated, although reflectorized signs seem as legible with a dark background.

7. Our observers represented better vision than the average, and therefore consideration was given to the poorer vision to be expected on the highway. A point corresponding to any given vision value can be obtained from the 20/20 curves presented. The distance values

shown may be of use to those who are setting standards of vision for drivers.

The distance values obtained indicate the use of at least 10-in. D or 12-in. B letters if 10 sec. warning is to be given the driver at 50 miles per hour at night.

8. The demonstration of a nonlinear relationship between letter size and distance is of importance if projection of values is attempted from tests on the smaller letter sizes. It has been generally assumed that such projections can be made on a linear basis. Our results indicate that the relationship which may appear linear in the smaller values, may be actually curvilinear. In this case the curve must be known in order to project.

9. In a previous paper by one of us, other factors of import to sign efficiency were derived, and it is to be hoped that studies will be made of these. It is probable that glance legibility will be somewhat shorter than the pure legibility values here reported and it is suggested that a correction factor will be derived eventually by which glance legibility may be obtained from the pure legibility values.

10. Analysis of sign efficiency in this general fashion will furnish a scientific basis for the development of highway signs to meet the increasing demands of modern highways.

The field observations were carried out under the Special Highway Division of the Pennsylvania Turnpike Commission with the cooperation of the Division under Mr. George Hitchcock, and were made possible by the interest and guidance of Mr. Charles M. Noble, Special Highway Engineer. We are indebted to Mr. Guy Kelcey and the Signal Service Corporation for loan of reflectors. Personnel was obtained through the fine cooperation of the officials of the Turnpike Commission, the officers of the Harrisburg Post of the Pennsylvania National Guard and the Commandant of the Army Medical Field Service School at Carlisle.

PHYSICAL SEPARATION OF OPPOSING STREAMS OF TRAFFIC

By Arnold H. Vey

Traffic Engineer, State of New Jersey

SYNOPSIS

The before and after accident experience on the dividing of a four-lane highway in New Jersey by a method of jacking apart the adjacent slabs presents pertinent data that are necessary in evaluating the benefits of divided highway construction.

The years of 1933 and 1934 are compared to 1937 and 1938 for the 10.64 miles of this section. There was a decrease in fatal accidents of 83.3 per cent, a decrease of 48.5 per cent in non-fatal injuries and a 17.6 per cent decrease in property damage accidents. The total reduction in accidents of all kinds was 40.4 per cent. Comparing these reductions to State-wide data for the same period, it is found that there was an increase of 6.7 per cent in total accidents, a decrease of 21.4 per cent in fatal accidents, and a 5.3 per cent decrease in non-fatal injury accidents. These changes occurred while the total traffic in the State was increasing 44 per cent as measured by the total gasoline consumption.

Further breakdowns of the data indicate that there was little change in total accidents at intersections, but a considerable reduction between intersections. It would seem that a wider dividing island would be a better accident preventive at intersections. The reduction of accidents was greater for the night than for the day, being 47.2 per cent at night and 31.4 per cent in the day.

The cost of this reconstruction was about $50,000 per mile. By applying National Safety Council figures to the reduction in accidents, it is found that savings in accident costs will pay for this conversion in slightly over 3 years.

Advanced thinking coached by continued and extensive researches in the causes and prevention of highway accidents has more and more convinced authorities that the "human side" of the accident problem can be effectively and economically dealt with through the application of appropriate roadway design principles—design features that lessen the importance of human failures—design features that make it less possible for drivers to have accidents because of improper driving practices.

A feature of highway design that has had a favorable trend in use in recent years is that of the divided roadway or dual type of highway.

The functional purpose of the divisional or center island is of course that of separating opposing flows of traffic, making it difficult or almost impossible for vehicles traveling in opposite directions to collide head-on. Some traffic authorities are of the opinion that the center island too might curtail other types of accidents such as collisions between vehicles traveling in the same direction, pedestrian accidents and others.

Have divided roadways brought about a decrease in accidents?

In the words of one of our more prominent citizens—"Let us look at the record."

In the year 1935, the New Jersey State Highway Department, convinced of the value of segregating opposing flows of traffic on heavily traveled four-lane highways, began a program of highway dualization along the important trans-state route between Trenton and Newark.

The first stretch of this roadway so converted has been completed long enough to indicate with some degree of accuracy the results in accident reduction which might be expected from highway dualization, other factors remaining the same.

The section concerned is that portion of State Highway Route No. 26 in the

Townships of North Brunswick, South Brunswick and Plainsboro, extending for a distance of 10.64 miles. The years 1935 and 1936 represent the construction period; 1933 and 1934 the "before" period and 1937 and 1938 the "after" period.

This section is substantially rural with little roadside development except occasional billboards, gasoline stations, garages and small buildings primarily used for commercial purposes. Intersection frequency is slightly more than one intersection per mile of roadway.

The highway cross section before the improvement included 39 ft. 6 in. of pavement, 16 ft. total shoulder area and 45 ft. sidewalk area, making a total of 100 ft. of right of way. After the improvement, which consisted in sliding one lane of concrete to the side for a sufficient distance to permit the construction of a 12-ft. island, the cross section included 24 ft. 6 in. of pavement each side of the 12-ft. center island, 10 ft. 6 in. total shoulder area and a 28-ft. 6-in. sidewalk area.

"BEFORE" AND "AFTER" ACCIDENT EXPERIENCE

A comparison of the number of accidents by severity "before" and "after" the conversion, gives the results in Table 1.

Because of the improvement in the general accident situation throughout the State in the past few years, particularly fatal accidents, it is possible that the accident experience along Route 26 in the years 1937 and 1938 would have improved more or less comparably to that of the entire State, assuming the roadway had remained as an undivided four-lane highway. On the other hand, if because of the conversion to a divided roadway, accident experience along Route 26 in the years 1937 and 1938 improved considerably over that for the entire State, it is reasonable to assume that the physical separation of opposing streams of traffic is largely responsible for the improvement.

Table 2 gives a comparison of the change in motor vehicle registration, gasoline consumption and accident experience in the years 1937 and 1938 over 1933 and 1934 for the entire State and the change in accident experience along the section of Route 26 concerned for the same period.

Comparable traffic counts along the section of Route 26 concerned for the "before" and "after" period are not

TABLE 1

ACCIDENT EXPERIENCE STATE HIGHWAY ROUTE NO. 26 BEFORE (1933 AND 1934) AND AFTER (1937 AND 1938) THE CONVERSION TO A DUAL HIGHWAY, ACCORDING TO SEVERITY

Accident Severity	No. Brunswick		So. Brunswick		Plainsboro		Total accidents		Change, %
	Before	After	Before	After	Before	After	Before	After	
Fatal	7	..	18	4	5	1	30	5	83.3 Dec.
Non-fatal injury	30	13	81	45	31	15	142	73	48.5 Dec.
Property damage	26	17	63	49	19	23	108	89	17.6 Dec.
Total accidents	63	30	162	98	55	39	280	167	40.4 Dec.
Change, %	52.4 Dec.		39.5 Dec.		29.1 Dec.		40.4 Dec.		

available. However, the increase in traffic volume in the 1937-1938 period over the 1933-1934 period, is equal to at least the increase in gasoline consumption in the entire State.

The effect of the conversion of this section of Route 26 to a divided highway on accident experience at "at and between intersection" locations and also the effect on accidents of different types, are shown in Table 3.

It is noted that the entire improvement along this stretch of roadway is confined to "between intersection" locations. This is undoubtedly due to the small number of intersections within this 10-mile area of highway and also due to the influence of factors of chance because of the low number of accidents at intersections. Possibly, too, because the center island is only 12 ft. wide, the influence as an accident preventive is not so great as if the island had been wider.

It is noted that accidents of all types, with the exception of right angle collisions, decreased along this stretch of roadway after the conversion. The percentages of decreases are indicated in the tabulation.

A detailed analysis of pedestrian acci-

TABLE 2

	Increase, %	Decrease, %
Motor vehicle registration	17.3	....
Gasoline consumption	44.0	....
Total accidents in state	6.7	....
Fatal accidents in state	....	21.4
Non-fatal injury accidents in state	....	5.3
Total accidents along Route 26	....	40.4
Fatal accidents along Route 26	....	83.3
Non-fatal injury accidents along Route 26	....	48.5

TABLE 3

ACCIDENT EXPERIENCE STATE HIGHWAY ROUTE 26 BEFORE (1933 AND 1934) AND AFTER (1937 AND 1938) THE CONVERSION TO A DUAL HIGHWAY, AT AND BETWEEN INTERSECTIONS

Type of accident	At intersection		Between intersection		Total		Change, %
	Before	After	Before	After	Before	After	
Right angle	8	11	2	2	10	13	30.0 Inc.
Same direction	13	21	129	86	142	107	24.6 Dec.
Opposite direction (left-turn)	3	2	1	...	4	2	50.0 Dec.
Opposite direction (head-on)	5	..	42	4	47	4	91.5 Dec.
Fixed object	2	2	28	20	30	22	26.6 Dec.
Pedestrian	1	..	18	5	19	5	73.7 Dec.
Other types	3	1	25	13	28	14	50.0 Dec.
Total	35	37	245	130	280	167	40.4 Dec.
Change, %	5.7 Inc.		46.9 Dec.		40.4 Dec.		

dents for both the "before" and "after" period indicates not only a decrease in pedestrian accidents when the pedestrian was crossing the highway but also in pedestrian accidents when the pedestrian was walking along the highway.

For the "before" period, 6 of the 19 pedestrian accidents concerned persons walking along the highway, while for the "after" period, 2 of the 5 accidents concerned pedestrians walking along the highway.

Table 4 gives the results of the conversion on accident experience separated by hours of daylight and hours of darkness.

TABLE 4

DAY AND NIGHT ACCIDENT EXPERIENCE STATE HIGHWAY ROUTE 26 BEFORE (1933 AND 1934) AND AFTER (1937 AND 1938) THE CONVERSION TO A DUAL HIGHWAY.

Type of accident	Day		Night		Total		Change, %
	Before	After	Before	After	Before	After	
Right angle	8	7	2	6	10	13	30.0 Inc.
Same direction	49	49	93	58	142	107	24.6 Dec.
Opposite direction (left-turn)	1	..	3	2	4	2	50.0 Dec.
Opposite direction (head-on)	20	3	27	1	47	4	91.5 Dec.
Fixed object	18	15	12	7	30	22	26.6 Dec.
Pedestrian	5	2	14	3	19	5	73.7 Dec.
Other types	20	7	8	7	28	14	50.0 Dec.
Total	121	83	159	84	280	167	40.4 Dec.
Change	31.4 Dec.		47.2 Dec.		40 4 Dec.		

ECONOMIC SAVING

The fundamental characteristics of dual highways result in favorable operating conditions, all of which have a definite bearing on the economics of this type of construction.

These include tangible items, the money value of which can at least be partially computed or estimated, such as smoother operation and accident reduction; and intangible items, the money value of which cannot be readily computed, such as decreased mental and physical exertion and irritation while driving, suffering and sorrow which inevitably accompany traffic accidents.

From the standpoint of accident reduction alone, the economic saving after a comparatively few years more than equals the initial cost of construction. For example, the approximate cost of the conversion of this section of roadway to a divided highway is approximately $50,000 per mile of road. The saving in accident costs, computed on the basis of the National Safety Council's figures of $11,500 for each death, $425 for each person injured and $125 for each property damage accident, amounts to $188,500 per year.

At this rate, the saving in accident costs resulting from the conversion of this roadway to a divided roadway will pay for the total cost of construction, plus interest charges at the rate of 6 per cent, in slightly more than three years.

CONCLUSIONS

It is apparent that the physical separation of opposing streams of traffic has a substantial, favorable effect on accident experience. Whenever it is physically and economically possible, therefore, highways of four or more lanes in width should be of the divided type.

PEDESTRIAN RESEARCH

By Burton W. Marsh

Director, Safety and Traffic Engineering Department
American Automobile Association

SYNOPSIS

Impressed with the seriousness of the plight of the pedestrian—the forgotten man in traffic—the American Automobile Association three years ago instituted a study of the subject.

While revealing many hitherto unrealized facts about pedestrian accidents and conditions, the study sharply emphasizes the inadequacy of available information. It points to the very great need for extensive further research and activity in such major areas as:

Fact-Gathering: In most localities present pedestrian accident reports fail principally to provide adequate information concerning acts, conditions, etc., immediately preceding the accident and relating to it.

Life Expectancy: Two out of three pedestrians killed are over 40, and the major toll is for those past 50. However, children of school age are the group which seems to warrant the greatest attention, because of their high injury rate (highest of any age-group) and the years of expected human life ahead. Our study indicated that total years of expected life lost reaches the highest point when children 5-to-14 are killed in traffic.

Increased Visibility of Crosswalks: Standing out sharply is the need for further studies of an engineering nature to assist pedestrians. Needed especially is a careful re-examination of pedestrian crosswalk design, particularly where located in the midblock or at irregularly shaped intersections. A large number of crosswalks in this country are virtually invisible to the motorist from a reasonable distance away. Instead of the usual 4- to 6-in. paint lines, the line at the near right side of the crosswalk should be about 24 in. wide. Some provision should be made to indicate the location of unusually placed crosswalks, especially at night. England has reduced its pedestrian toll by the use of the Belisha Beacon. A partial study in London shows that fatalities decreased 25 per cent and injuries 12 per cent in a before-and-after study.

Intoxication of Pedestrians: Studies in New York City and Cuyahoga County, Ohio, of brain tissue and body fluids for presence of alcohol indicate that this may be a much more important factor in other areas than is believed. These showed two out of five pedestrians killed had been drinking substantial amounts of alcoholic beverages.

Non-Drivers Being Killed: Pedestrians who do not drive are principally those being killed. Studies in Connecticut corroborated by five similar studies, indicated that nine out of ten of those killed did not have a drivers license. But we do not know well enough just what kinds of knowledge they lack—what wrong ideas they may have regarding motor operation and vehicular traffic.

Pedestrian Invisibility: We know all too little about the point which stands out so very prominently in so many pedestrian accidents—the fact that the driver did not see the pedestrian or did not see him until the instant when he hit him.

Recently I had luncheon in New York with the Director of Public Relations of one of our large industrial organizations. He had become keenly interested in pedestrian traffic problems and had thought of engaging in some activities to help improve pedestrian conditions. His proposal to the President of the Company was greeted with decided derision. "Why in the world should we devote attention to the pedestrian? He isn't of any importance in the accident toll" was the gist of the reaction. But he changed his mind when he found out that in cities two-thirds and often more of the persons killed in traffic were pedestrians, and that even in rural areas, pedestrians account for one-quarter to one-third of the total.

Three years ago the American Automobile Association impressed with the seriousness of the plight of the forgotten-man-in-traffic, instituted a study of the subject. Aided by grants from the Automotive Safety Foundation, the Association engaged Professor William J. Cox, then of Yale University, to make the original studies. Since Mr. Cox became Highway Commissioner of Connecticut, the study has been carried on at the Association's headquarters. Originally compiled in a technical report, the material has been formulated into a semi-popular booklet, "Pedestrian Protection."

Some of the main findings of this three-year study are as follows:

1. *Elderly:* Two out of three pedestrians killed are over 40 and the major toll is for those past 50, who are five times as likely as school children to be killed while walking on our streets and highways.
2. *Male:* About seven out of ten of the pedestrians killed are male.
3. *Death After Dark:* The majority of pedestrians killed are meeting death after dark. The month of December, with fewest hours of daylight, is often the deadliest month for those afoot.
4. *Non-Drivers:* Nine out of ten or more of the pedestrians killed do not drive a car.
5. *Midblock Crossings:* Three out of five pedestrian fatalities in cities occurred away from intersections. Many involve coming out from between parked cars—a practice against which the motorist is almost powerless.
6. *Ingested Alcohol:* Studies by the Chief Medical Examiner of New York City and the Coroner of Cuyahoga County, Ohio (Cleveland area) indicate that two out of five pedestrians killed had been drinking substantial amounts of alcoholic beverages. Saturday and Sunday account for the largest percentage of pedestrian drinking.
7. *Under-Privileged and Low-Income Groups:* Spotting fatalities by locations of residence of pedestrians involved, in Dallas, Washington, D. C., Detroit and other places has indicated that certain groups have unusually high numbers of pedestrian fatalities. These are generally groups like the foreign-born, non-English speaking, low-income and those more poorly endowed mentally.
8. *School-Aged Children:* The fatality rate for children of school age is not high. However, the injury rate in traffic is the highest of all age-groups in the data studied.

REASONS FOR BAD RECORD

The natural reaction of the student is to seek reasons behind his findings, for an understanding of *why* is always helpful in deciding upon what to do to correct bad conditions. The following points seem to be significant.

Unfamiliarity with Driving Problems: Middle-aged and older persons formed their walking habits and their attitudes before there was any considerable automobile traffic problem. They are unfamiliar with motor vehicle operation and the problems of driving. They are often unwilling to face the need of readjustments in their walking practices.

Relative Invisibility of Pedestrians: Ignorance by a great majority of pedestrians of their relative invisibility on the highway at night is probably one of the major causes for night-time pedestrian fatalities. The headlights look so very bright to the pedestrian that he is sure

that the driver can see him long before the driver actually does. Over and over again in the special case history study of 1,715 fatal accidents,[1] the driver asserted that he didn't see the pedestrian or didn't see him soon enough.

A companion point is the failure of the average motorist to realize that the pedestrian does not have a proper understanding of this matter, and the failure of motorists to take proper corrective steps themselves (including keeping headlights in better condition, keeping windshields clean and traveling considerably slower at night).

Effects of Advancing Years: Believed to be of considerable importance are the less keen vision and hearing, less agile pace, and lowered alertness in advancing years. Lowered resistance is also important as will be shown later. Momentary lapses of attention and unawareness of existing hazards are likewise important factors.

Unwise Walking Practices: Studies indicate clearly the predominance of unsound walking practices on the part of pedestrians. We greatly need general acceptance of a code for cooperative street use by the motorists and pedestrians which really fits today's conditions.

Lack of Proper Pedestrian Facilities: One major reason why pedestrians have trouble today is that street and traffic facilities have not been properly designed for them.

Lack of Facts: In most localities there is a serious lack of facts concerning pedestrian conditions.

RECENT STUDIES

The Connecticut study of 1,031 pedestrian fatalities which showed that over nine out of ten of those of licensable age had never been licensed to drive aroused very great interest in this factor. Five other rather similar studies have corroborated this significant point. These are shown in Table 1.

TABLE 1

PERCENTAGE OF PERSONS KILLED AS PEDESTRIANS WHO WERE WITHIN LICENSABLE AGE, BUT WERE NOT LICENSED

	Percent
Pedestrians killed in New Jersey in 1938, who were eligible for driver's license, but did not have such a license.....	93
Pedestrians 16 years and over, in Pennsylvania, in 1938 who were not licensed drivers. (In first six months of 1939, this group was 96.4 percent.).....	97.2
Pedestrians killed in South Carolina from November, 1937, through May, 1939, of licensable age who were never licensed to drive in South Carolina.....	85.0
Pedestrians killed in Connecticut in 1938, of licensable age, who did not have such a license.....	92.0
Pedestrians killed in Washington, D. C., based on 8-year average (1931-1938) who were of licensable age who did not have nor had had a license since 1926.	90.0

There are three deficiencies in some of these studies which should be noted:

1. Some of the persons not licensed in the jurisdiction studied may have been licensed elsewhere;
2. In some of the studies the records either were not checked back very far or records were not available to check back to find if the person had ever been licensed to drive;
3. In some places the person may have driven before there was any licensing law or records of persons licensed.

The South Carolina study yields some other interesting information, shown in Table 2.

[1] "A Case History Study of Fatal Accidents," this volume, p. 362.

Relative Emphasis Warranted on Aiding the Elderly

The facts point clearly to the high number of pedestrian fatalities involving persons of middle age and over. No one will question the importance of seeking to reduce that toll. However, there are other aspects to the matter which deserve study and consideration. Mr. Earl Allgaier of the American Automobile Association has made several interesting analyses which bear importantly on the questions of the degree of emphasis in corrective measures which should be devoted to various age groups.

TABLE 2

PEDESTRIAN INFORMATION FROM SOUTH CAROLINA STUDY

NOVEMBER 1937 TO MAY 1939

266 pedestrians killed.
180 of these were over 14 and residents of the state.

Unlicensed	Percent
Males	82
Females	97
Whites	76
Colored	95
White males	73
Colored males	94
White females	92
Colored females	100

Ratio of Injuries to Fatalities

Figure 1 brings out a very significant point. It is based on a study of records of 17 states for the years 1936, 1937 and 1938 involving 16,726 pedestrians who were killed, and 205, 562 pedestrians who were injured. On the average, 12.3 pedestrians were injured for each one killed. However, in the school age group of 5-14, the ratio is 29.8, and after that age group the ratio steadily drops off to a low of 3.6 for persons aged 65 and over.

This helps to answer why the death rate among elderly pedestrians is so high. Having been involved in an accident they do not have the recuperative power which younger persons have. From the point of view of pedestrian casualties, major attention needs to be devoted to the school age children.

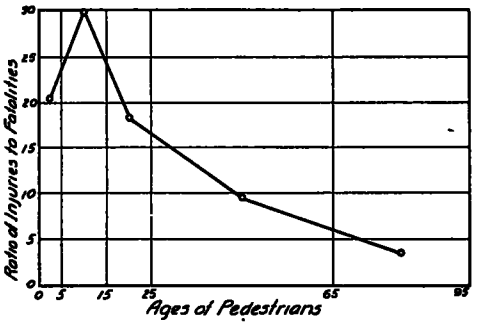


Figure 1. Ratios of Pedestrian injuries to fatalities at various ages. Data from 17 states from 1936 to 1938. 205,562 injured, 16,726 killed.

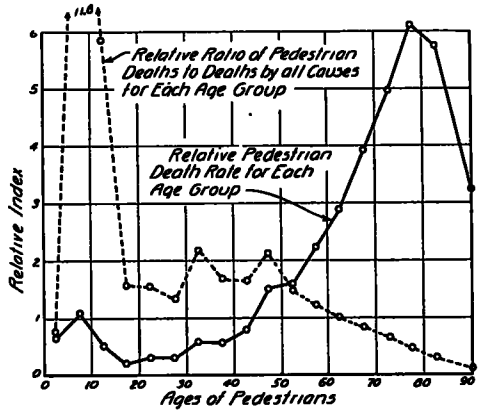


Figure 2. Pedestrian Deaths by Age Groups. From "A Case History Study of Fatal Accidents," p. 362.

Comparison of Pedestrian Deaths to Deaths by All Causes

Figure 2 shows a comparison of the relative death rates for pedestrians of various ages. It also shows the ratio of the comparative pedestrian death rates for each age group to a comparably arranged index for death by all causes for

each age group. This study, likewise prepared by Earl Allgaier, is based on the data from the special case history study of 1,715 fatalities.[1]

Compared to all causes of death, traffic is the most serious for the age group from 5-9, where nearly 12 times as many pedestrians are killed as would be expected if the pedestrian death rate curve followed the curve for deaths from all causes.

It will be quickly pointed out that there is no essential reason why pedestrian rates should be expected to follow the death rate from all causes, nor is this the assumption behind these curves. They are intended merely to point out where the sharpest divergence between the two death rate curves occur. Probably the principal merit of this analysis is to show the very high degree to which traffic hazards are the greatest hazards to life for children of grade school age.

Years of Expected Life Study

Figure 3 provides a further interesting angle on the costliness of pedestrian fatalities in terms of years of human life probably lost. Assume that the records of a typical thousand pedestrians killed in traffic are examined. Assume that the pedestrians are grouped by age and that the number in each age group is multiplied by the years of expected life for that age group, the total of all such multiplied sums for all the age groups would give the total number of years of expected life which had been lost because of the fatalities. The percentages of this total could then easily be figured for each age group. This curve shows those percentages.

Note that the percentage of the total years of expected life lost reaches the highest point in the age group 5-9, with 22.9 per cent; the age group zero to 4 is next most important, with a percentage of 13.1; and the 10-14 age group is next with 9.4 per cent.

Between the ages of 15 and 30, the percentages reach a low point for persons under 65. For persons over 65 the percentages decrease sharply. This curve again emphasizes the importance of attention to the child in traffic safety work.

This analysis, made by Mr. Earl Allgaier, is based upon records of 983 pedestrian fatalities, taken from the previously mentioned case history study of fatal accidents.

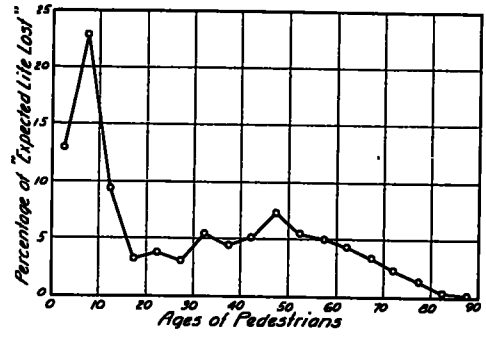


Figure 3. Years of expected life lost by Pedestrian Deaths for each age. 983 Pedestrian Deaths, Expected Life from U. S. Census. From "A Case History Study of Fatal Accidents," p. 362.

Ordinances Providing for Pedestrian Observance of Stop-and-Go Signals

Much interest has been generated recently concerning the desirability of legislation requiring pedestrians to obey traffic control signals. An inquiry among cities of over 50,000 population by the American Automobile Association produced 103 replies. Analysis provides the interesting information shown in Table 3.

It is interesting to note that while a substantial number of cities have ordinances requiring pedestrians to obey traffic control signals, only slightly over half enforce the law and less than half report good observance. Further study is warranted as to the reasons why good

observance is secured in some cities and not in others. Among probable reasons are failure to arrange signal locations, design and operation to suit pedestrian needs (for example, too long intervals, no signal ahead of the pedestrian as he crosses in certain directions, etc.), failure to convince the public that the measure is desirable, through a proper program of education, and improper handling of enforcement.

TABLE 3

ANALYSIS OF REPLIES RECEIVED FROM 103 CITIES REPLYING TO INQUIRY SENT OUT BY THE AAA CONCERNING ORDINANCES REQUIRING PEDESTRIAN OBSERVANCE OF STOP-AND-GO SIGNALS

No. of cities over 50,000 population replying	103
No. of cities requiring pedestrian observance of stop-and-go signals	65
No. of cities which reported enforcing the law	37
No. of cities reporting good observance	31
No. of cities reporting fair observance	20
No. of cities reported prohibiting "jaywalking"	33
No. of cities reporting good observance	9
No. of cities reporting fair observance	15

Crosswalk Design

Our studies lead us to believe that there is need for careful re-examination of the design of pedestrian crosswalks. Particularly where crosswalks are located in the midblock, or at irregular-shaped intersections, it is of very great importance that the motorist always be able to know in advance where the crosswalk is at which he is expected to yield the right-of-way to pedestrians. Yet, a large number of crosswalks in this country do not satisfy this reasonable requirement. For example, the usual 4- to 6-in. paint lines are virtually invisible from a reasonable distance back of the crosswalk location and from the height at which the motorist is expected to see it. At the near-right side of the crosswalk, if the customary type of paint line is used, it should be in the neighborhood of 24 in. wide, except where vehicle speeds are very slow. Furthermore, if snow or ice covers the markings, there is no way at all for a motorist to know of the existence of an irregular-marked crosswalk, although the pedestrian may be expecting the motorist to yield right-of-way there. At night many of the crosswalks are virtually invisible.

In England, this has been faced and the Belisha Beacon is one of the important results. Named after the then Head of the Ministry of Transport Mr. Hore-Belisha, it consists of a post with alternate dark and light bands on it and is surmounted by a large orange-colored globe which is lighted at night. By November 1, 1937, 28,000 of these special crossings had been established, 12,000 of which were in London, and evidences were that they were helping to reduce the pedestrian toll. Fatalities had decreased 25 per cent and injuries 12 per cent, in a partial before and after study. These beacons serve both day and night to help the motorists identify places at which they are to yield the right-of-way. There is a real need for much more effective crosswalk design in this country and it is especially important at irregularly located crosswalks.

FURTHER RESEARCH NEEDED

There is great need for extensive further research on pedestrian problems. A new type pedestrian island has produced an undue number of injuries. What's wrong with the design or with the plan governing its installation and public education concerning its presence?

Pedestrian accident facts are very inadequate. *They principally fail to provide adequate information as to acts, conditions, etc., immediately preceding the accident and relating to it.* In pedestrian fatality cases, unless there is an interested witness, the only source of in-

formation is the motorist. Honest as he may be, he is giving the viewpoint of the man-behind-the-wheel. Much more of the viewpoint of the man-on-foot is needed. In order to obtain this, we are advocating case studies of accidents involving pedestrians injured non-fatally. An interview form has been prepared. Another interesting proposal for gaining such information is that of Mr. Peter Stupka for pedestrian reporting of accidents.

The New York and Cuyahoga County studies of brain tissue and body fluids for presence of alcohol indicate that this may be a much more important factor in other areas than is believed. Further research should be carried on in a number of additional areas to find out the facts. One procedure would be for coroners to make it a matter of normal practice to test the body fluids of accident victims for the presence of alcohol. Another procedure would be for hospitals to make such tests where patients are willing and conditions are appropriate.

Many further studies of relationships between factors in pedestrian accidents are needed. For example, we know of the predominance of persons past 50 in the fatality toll, but we do not know enough about what those persons were doing—whether they were carrying anything light or anything white, where they were crossing, whether they looked or not, whether they have serious vision deficiencies or other physical disabilities. Similarly, we know that the persons who do not drive are principally those being killed, but we do not know well enough just what kinds of knowledge they principally lack—what wrong ideas they may have regarding motor vehicle operation and traffic. We do not know what are the most frequent bad practices of that particular group.

We know all too little regarding the effectiveness of various types of corrective measures, particularly in the field of education where there is great need for appraising the value of various plans, methods, etc.

We know all too little about the point which stands out so very prominently in so many pedestrian accidents—the fact that the driver did not see the pedestrian or did not see him until almost the instant when he hit him. While these assertions are partly rationalizations, they are not altogether so, and the whole problem warrants much further attention.

DISPERSION OF HIGHWAY TRAFFIC VOLUME BY TIME PERIODS

Nine Stations in Michigan

By W. Arthur Shelton

Senior Transportation Economist, Public Roads Administration

SYNOPSIS

The relative dispersion of highway traffic for the hours from 9 to 10 in the morning to 5 to 6 in the afternoon is much less than for the remaining hours of the day. This day period is also convenient for traffic counting, and the greater than average constancy of flow of traffic makes increased precision of estimate feasible. In resort areas and other areas of wide dispersion on week-end days, it may also be desirable to use only midweek days for estimation of traffic volume. Among the months, June, July, and August have the most nearly constant flow of traffic, but among the other months there is little choice for sampling periods.

This section of the analysis of the problem of estimating highway traffic volume includes the data at nine stations selected for the Highway Planning Survey of Michigan as representative of traffic in the Upper Peninsula and in the western part of the Lower Peninsula. The nine points include three near Humboldt in the Upper Peninsula and six in the western part of the Lower Peninsula, and they embrace some of the more widely dispersed populations of Michigan.

While the traffic population, or annual universe, of 8,784 hours at each of these points has a dispersion, or scatter, independent of other points, the locations were selected as characteristic of the several types of stations of the area. The dispersion by hours here set forth for time periods is an extension of similar studies of Hudson River crossings and State and local stations in Iowa presented to the Highway Research Board in previous years.[1]

The data consist of manual counts by hours for each point for a year ending on some date in January 1937 and including 366 days. The dispersion of the volume of traffic by hours is shown for the hours of the day, the days of the week, and the months of the year in nine tables of the standard deviation and nine of the coefficient of variation. The coefficient of variation has been charted as

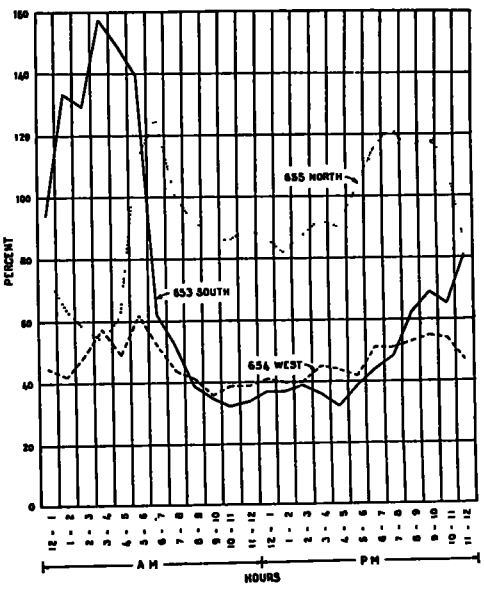


Figure 1. Coefficient of Variation of Highway Traffic Volume, by Hours, for Year 1936-37

indicating the relative dispersion in hourly volume of traffic among the subpopulations.

HOURS OF THE DAY

Figure 1 shows that during the day period from seven in the morning to five

[1] *Proceedings*, Highway Research Board, Vol. 14, 1934, pp. 399-410; Vol. 16, 1936, pp. 239-252; Vol. 17, 1937, pp. 413-419; Vol. 18, 1938, pp. 347-358.

in the afternoon the relative variation in total traffic volume among the hours of the day is smaller than for the remainder of the 24 hours. Samples could therefore be confined to the day period with a reduction in error of estimate. For the in the day period than during the remaining hours of the 24, as is shown for stations in the Lower Peninsula.

Figure 2 also manifests the general law of greater constancy during the day period from eight to six than during the

TABLE 1

STANDARD DEVIATION OF HIGHWAY TRAFFIC VOLUME, BY HOURS, FOR THE YEAR 1936-37[1], IN CHRONOLOGICAL ORDER

Hour	Michigan highway survey station numbers 650-655								
	650 East	650 South	650 West	651 North	651 East	652 South	653 South	654 West	655 North
	Number of vehicles								
a. m.									
12–1	13	8	9	6	6	20	5	27	78
1–2	11	7	7	5	5	20	5	17	54
2–3	6	4	5	4	6	13	2	14	40
3–4	4	2	3	3	4	11	2	13	32
4–5	3	2	4	3	3	10	2	11	34
5–6	5	3	4	6	4	15	2	18	78
6–7	10	6	7	12	10	27	4	24	120
7–8	13	6	10	17	12	36	7	30	123
8–9	16	7	13	21	16	46	6	40	149
9–10	21	9	18	21	19	60	7	37	173
10–11	28	11	24	24	20	66	7	47	196
11–12	28	11	25	25	20	65	7	48	218
p. m.									
12–1	27	10	24	23	18	57	7	48	201
1–2	30	12	25	23	20	57	8	51	193
2–3	40	15	36	26	21	64	9	57	238
3–4	40	15	36	26	20	70	9	68	265
4–5	39	14	36	28	23	70	9	85	297
5–6	41	15	37	30	25	72	11	72	326
6–7	40	15	34	24	19	62	11	74	334
7–8	44	17	36	26	19	67	11	69	329
8–9	43	16	37	25	20	64	11	62	269
9–10	34	13	29	20	16	50	10	54	231
10–11	24	10	19	21	19	37	7	41	177
11–12	16	8	12	10	9	27	7	28	100

[1] The year extends from near the middle of January, 1936, to the date before that of 1937 and includes 366 days for each station leg because of leap year.

more widely dispersed hours of the 24, an hourly pattern composed of a larger number of hours can be used to convert the small sample from the more nearly constant period to a full day. It is significant that even the highly variable traffic of the Upper Peninsula shows the same tendency toward greater constancy remainder of the 24 hours. These three points are located in the northwest section of the Lower Peninsula and represent both sparse and rather dense volume of traffic in areas of both forest and farm. The afternoon scatter is less than that of the morning, while in Figure 3 the opposite is true. The explanation is that in

TABLE 2

STANDARD DEVIATION OF HIGHWAY TRAFFIC VOLUME, BY HOURS, FOR THE YEAR 1936-37, IN ASCENDING ORDER

Michigan highway survey station numbers 650-651

Hour	650 East	Hour	650 South	Hour	650 West	Hour	651 North	Hour	651 East
	Number of Vehicles								
4–5 a.	3	4–5 a.	2	3–4 a.	3	3–4 a.	3	4–5 a.	3
3–4 a.	4	3–4 a.	2	4–5 a.	4	4–5 a.	3	3–4 a.	4
5–6 a.	5	5–6 a.	3	5–6 a.	4	2–3 a.	4	5–6 a.	4
2–3 a.	6	2–3 a.	4	2–3 a.	5	1–2 a.	5	1–2 a.	5
6–7 a.	10	6–7 a.	6	1–2 a.	7	12–1 a.	6	12–1 a.	6
1–2 a.	11	7–8 a.	6	6–7 a.	7	5–6 a.	6	2–3 a.	6
7–8 a.	13	8–9 a.	7	12–1 a.	9	11–12 p.	10	11–12 p.	9
12–1 a.	13	1–2 a.	7	7–8 a.	10	6–7 a.	12	6–7 a.	10
11–12 p.	16	12–1 a.	8	11–12 p.	12	7–8 a.	17	7–8 a.	12
8–9 a.	16	11–12 p.	8	8–9 a.	13	9–10 p.	20	8–9 a.	16
9–10 a.	21	9–10 a.	9	9–10 a.	18	8–9 a.	21	9–10 p.	16
10–11 p.	24	12–1 p.	10	10–11 p.	19	9–10 a.	21	12–1 p.	18
12–1 p.	27	10–11 p.	10	10–11 a.	24	10–11 p.	21	9–10 a.	19
10–11 a.	28	10–11 a.	11	12–1 p.	24	12–1 p.	23	6–7 p.	19
11–12 a.	28	11–12 a.	11	11–12 a.	25	1–2 p.	23	7–8 p.	19
1–2 p.	30	1–2 p.	12	1–2 p.	25	10–11 a.	24	10–11 p.	19
9–10 p.	34	9–10 p.	13	9–10 p.	29	6–7 p.	24	10–11 a.	20
4–5 p.	39	4–5 p.	14	6–7 p.	34	11–12 a.	25	11–12 a.	20
2–3 p.	40	3–4 p.	15	2–3 p.	36	8–9 p.	25	1–2 p.	20
3–4 p.	40	6–7 p.	15	3–4 p.	36	2–3 p.	26	3–4 p.	20
6–7 p.	40	2–3 p.	15	4–5 p.	36	3–4 p.	26	8–9 p.	20
5–6 p.	41	5–6 p.	15	7–8 p.	36	7–8 p.	26	2–3 p.	21
8–9 p.	43	8–9 p.	16	5–6 p.	37	4–5 p.	28	4–5 p.	23
7–8 p.	44	7–8 p.	17	8–9 p.	37	5–6 p.	30	5–6 p.	25

TABLE 3
STANDARD DEVIATION OF HIGHWAY TRAFFIC VOLUME, BY HOURS, FOR THE YEAR 1936-37, IN ASCENDING ORDER

Michigan highway survey station numbers 652-655							
Hour	652 South	Hour	653 South	Hour	654 West	Hour	655 North
	Number of vehicles						
4–5 a.	10	2–3 a.	2	4–5 a.	11	3–4 a.	32
3–4 a.	11	3–4 a.	2	3–4 a.	13	4–5 a.	34
2–3 a.	13	4–5 a.	2	2–3 a.	14	2–3 a.	40
5–6 a.	15	5–6 a.	2	1–2 a.	17	1–2 a.	54
12–1 a.	20	6–7 a.	4	5–6 a.	18	12–1 a.	78
1–2 a.	20	12–1 a.	5	6–7 a.	24	5–6 a.	78
6–7 a.	27	1–2 a.	5	12–1 a.	27	11–12 p.	100
11–12 p.	27	8–9 a.	6	11–12 p.	28	6–7 a.	120
7–8 a.	36	7–8 a.	7	7–8 a.	30	7–8 a.	123
10–11 p.	37	9–10 a.	7	9–10 a.	37	8–9 a.	149
8–9 a.	46	10–11 a.	7	8–9 a.	40	9–10 a.	173
9–10 p.	50	11–12 a.	7	10–11 p.	41	10–11 p.	177
12–1 p.	57	12–1 p.	7	10–11 a.	47	1–2 p.	193
1–2 p.	57	10–11 p.	7	11–12 a.	48	10–11 a.	196
9–10 a.	60	11–12 p.	7	12–1 p.	48	12–1 p.	201
6–7 p.	62	1–2 p.	8	1–2 p.	51	11–12 a.	218
2–3 p.	64	2–3 p.	9	9–10 p.	54	9–10 p.	231
8–9 p.	64	3–4 p.	9	2–3 p.	57	2–3 p.	238
11–12 a.	65	4–5 p.	9	8–9 p.	62	3–4 p.	265
10–11 a.	66	9–10 p.	10	3–4 p.	68	8–9 p.	269
7–8 p.	67	5–6 p.	11	7–8 p.	69	4–5 p.	297
3–4 p.	70	6–7 p.	11	5–6 p.	72	5–6 p.	326
4–5 p.	70	7–8 p.	11	6–7 p.	74	7–8 p.	329
5–6 p.	72	8–9 p.	11	4–5 p.	85	6–7 p.	334

TABLE 4

COEFFICIENT OF VARIATION OF HIGHWAY TRAFFIC VOLUME, BY HOURS, FOR THE YEAR 1936-37, IN CHRONOLOGICAL ORDER

Hour	Michigan highway survey station numbers 650-655								
	650 East	650 South	650 West	651 North	651 East	652 South	653 South	654 West	655 North
	Percent that standard deviation is of mean for each hour								
a. m.									
12–1	83.33	91.09	85.77	88.85	81.04	72.25	94.25	45.31	74.08
1–2	98.18	111.09	104.09	111.41	100.73	87.66	133.17	42.13	63.72
2–3	96.48	100.07	106.26	122.14	134.56	84.63	129.23	48.60	57 54
3–4	93.94	97.15	108.11	139.28	123.27	88.25	157.06	57.32	53.69
4–5	63.56	78.07	92.92	145.98	108.66	89 59	148.03	48.91	61.59
5–6	53.50	46.68	68.70	116.45	99.01	84 53	139.41	61.90	111.88
6–7	46.15	45.30	56.03	74.77	69.03	63.51	62.36	51.87	125.15
7–8	44.79	52.14	45.97	53.12	58.52	57.37	52.39	43.74	100.56
8–9	45.08	43.39	46.29	58.55	58.91	58.12	39.05	40.79	91 64
9–10	44.48	42.37	51.63	49.45	55.26	59.85	35.23	35.51	88.89
10–11	50.41	42.70	59.13	50.65	53.16	57.55	32.48	39.36	86.36
11–12	53.84	46.18	60.85	49.71	53.53	54.30	34.35	38.79	89.10
p. m.									
12–1	54.18	43.28	63 11	47.35	50.00	48.60	36.59	41.06	86.76
1–2	49.85	43 66	55 51	48.55	48.57	47.38	37.32	39.92	81.75
2–3	59.36	53 07	69.30	51.28	49 94	47.02	38.80	40 01	87.03
3–4	56.54	46.77	63.14	49.65	48.95	49.54	36.13	45.19	91.58
4–5	50.40	44.11	60.08	40.33	45.00	45 22	32 37	43.51	89.89
5–6	58.65	44.98	70.65	43.58	49.22	48.21	38.55	42.06	102.33
6–7	62.37	48.50	70.70	48.40	48.13	52.09	44.00	51.14	114.70
7–8	75.35	66.10	78 14	57.49	59.13	58.66	48.32	51.47	122.42
8–9	90.44	74.76	96.57	65.78	71.32	65.18	61.89	52.96	115.34
9–10	91.07	72.68	100.50	73.08	75.85	65.55	68.62	54.86	119.34
10–11	85.22	72.46	93.50	105.77	107.91	65.12	64.95	54.47	112.84
11–12	76.39	73.76	81.96	93 76	87.54	67.07	80.81	46 82	84.98

TABLE 7

STANDARD DEVIATION OF HIGHWAY TRAFFIC VOLUME, FOR HOURS, FOR THE YEAR 1936-37, BY DAY OF WEEK, IN CHRONOLOGICAL ORDER

Michigan highway survey station numbers 650-655

Day	650 East	650 South	650 West	651 North	651 East	652 South	653 South	654 West	655 North
	Number of vehicles								
Sunday	54	20	47	32	23	86	15	102	350
Monday	29	12	23	27	20	63	10	61	242
Tuesday	26	11	21	24	19	52	10	58	100
Wednesday	29	12	24	27	21	54	10	54	98
Thursday	28	12	23	26	19	57	10	61	98
Friday	30	13	24	29	24	66	11	64	172
Saturday	37	16	30	32	25	77	12	63	251

TABLE 8

STANDARD DEVIATION OF HIGHWAY TRAFFIC VOLUME, FOR HOURS, FOR THE YEAR 1936-37, BY DAY OF WEEK, IN ASCENDING ORDER

Michigan highway survey station numbers 650-651

Day	650 East	Day	650 South	Day	650 West	Day	651 North	Day	651 East
	Number of vehicles								
Tuesday	26	Tuesday	11	Tuesday	21	Tuesday	24	Tuesday	19
Thursday	28	Monday	12	Monday	23	Thursday	26	Thursday	19
Monday	29	Wednesday	12	Thursday	23	Monday	27	Monday	20
Wednesday	29	Thursday	12	Wednesday	24	Wednesday	27	Wednesday	21
Friday	30	Friday	13	Friday	24	Friday	29	Sunday	23
Saturday	37	Saturday	16	Saturday	30	Saturday	32	Friday	24
Sunday	54	Sunday	20	Sunday	47	Sunday	32	Saturday	25

TABLE 9

STANDARD DEVIATION OF HIGHWAY TRAFFIC VOLUME, FOR HOURS, FOR THE YEAR 1936-37, BY DAY OF WEEK, IN ASCENDING ORDER

Michigan highway survey station numbers 652-655

Day	652 South	Day	653 South	Day	654 West	Day	655 North
	Number of vehicles						
Tuesday	52	Monday	10	Wednesday	54	Wednesday	98
Wednesday	54	Tuesday	10	Tuesday	58	Thursday	98
Thursday	57	Wednesday	10	Monday	61	Tuesday	100
Monday	63	Thursday	10	Thursday	61	Friday	172
Friday	66	Friday	11	Saturday	63	Monday	242
Saturday	77	Saturday	12	Friday	64	Saturday	251
Sunday	86	Sunday	15	Sunday	102	Sunday	350

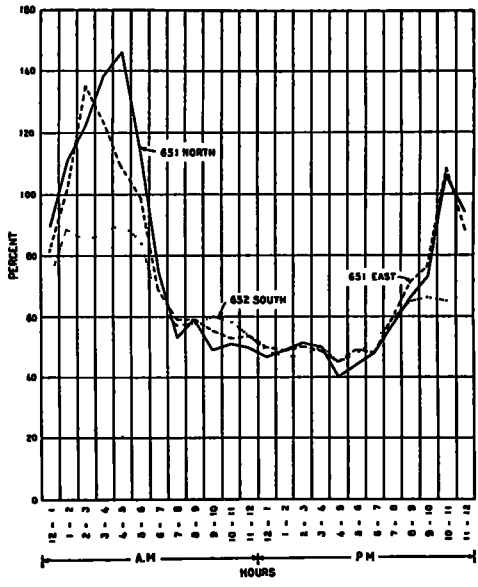


Figure 2. Coefficient of Variation of Highway Traffic Volume, by Hours, for Year 1936-37

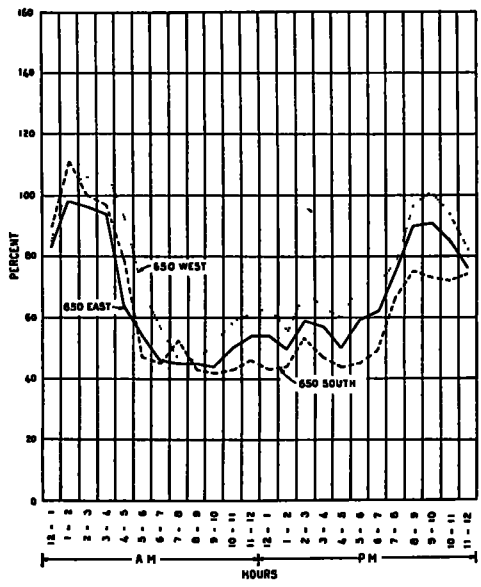


Figure 3. Coefficient of Variation of Highway Traffic Volume, by Hours, for Year 1936-37

TABLE 10

COEFFICIENT OF VARIATION OF HIGHWAY TRAFFIC VOLUME, FOR HOURS, FOR THE YEAR 1936-37, BY DAY OF WEEK, IN CHRONOLOGICAL ORDER

Day	Michigan highway survey station numbers 650-655								
	650 East	650 South	650 West	651 North	651 East	652 South	653 South	654 West	655 North
	Percent that standard deviation is of mean of each hour of day								
Sunday	95.30	80.24	104.42	94.85	88.55	85.27	77.01	81.04	110.73
Monday	81.33	68.81	86.26	84.66	82.24	84.40	72.78	69.42	135.52
Tuesday	82.99	75.73	88.04	83.25	83.20	77.41	75.94	68.65	76.74
Wednesday	85.61	74.87	93.08	85.42	87.33	77.62	75.58	63.29	74.60
Thursday	81.32	71.99	86 76	85.10	81.21	77.62	73.57	67.32	72.68
Friday	80.14	72.62	84.96	87.67	90.18	80.00	75.46	67.31	99.65
Saturday	83.37	79.68	90 84	83 51	81.56	77.37	70.48	61.73	109.48

the resort area of the Upper Peninsula the dispersion increases after 10 in the morning. The night peaks of dispersion are not so pronounced at 652 South as at the other two points on Figure 2, and the explanation is that this State highway station near Cadillac has a more nearly constant night flow than the points on the routes of smaller volume.

While station 653 South and 654 West show something of the usual night peaks of dispersion, that of 655 North shows a valley from 12 midnight to five in the morning. Nevertheless the level plane from nine to five in the day period standing between two peaks, as shown on Figure 1, is clearly marked for 655 North. This station is located between Chicago

and the Lake Michigan resort area in Michigan and also on the route of fruit and vegetable movement, and both of these types of traffic occur in widely varying volume. This station should represent the most extreme variation from the normal law of constancy of dispersion during the day period; and yet it shows markedly the day plane (from 9 a.m. to 5 p.m.), with a much wider coefficient of variation, however, than have the other eight stations. This curve (655 North) seems to indicate that the tendency toward small dispersion during the day period is very general if not universal. It shows that the relative dispersion is materially less from nine in the morning to five in the afternoon.

DAYS OF THE WEEK

The dispersion of the hours by days of the week is roughly 80 percent of the mean, while that of the hours between nine in the morning and five in the afternoon for all days of the week is roughly only 50 percent. This contrast is due to the inclusion of the widely dispersed hours in the 24 hours in the former case. The other striking feature of the dispersion by days of the week is that the

TABLE 11

COEFFICIENT OF VARIATION OF HIGHWAY TRAFFIC VOLUME, FOR HOURS, FOR THE YEAR 1936-37, BY DAY OF WEEK, IN ASCENDING ORDER

Day	Michigan highway survey station numbers 650-651								
	650 East	Day	650 South	Day	650 West	Day	651 North	Day	651 East
	Percent that standard deviation is of mean of each day of week								
Friday	80.14	Monday	68.81	Friday	84.96	Tuesday	83.25	Thursday	81.21
Thursday	81.32	Thursday	71.99	Monday	86.26	Saturday	83.51	Saturday	81.56
Monday	81.33	Friday	72.62	Thursday	86.76	Monday	84.66	Monday	82.24
Tuesday	82.99	Wednesday	74.87	Tuesday	88.04	Thursday	85.10	Tuesday	83.20
Saturday	83.37	Tuesday	75.73	Saturday	90.84	Wednesday	85.42	Wednesday	87.33
Wednesday	85.61	Saturday	79.68	Wednesday	93.08	Friday	87.67	Sunday	88.55
Sunday	95.30	Sunday	80.24	Sunday	104.42	Sunday	94.85	Friday	90.18

TABLE 12

COEFFICIENT OF VARIATION OF HIGHWAY TRAFFIC VOLUME, FOR HOURS, FOR THE YEAR 1936-37, BY DAY OF WEEK, IN ASCENDING ORDER

Day	Michigan highway survey station numbers 652-655						
	652 South	Day	653 South	Day	654 West	Day	655 North
	Percent that standard deviation is of mean of each day of week						
Saturday	77.37	Saturday	70.48	Saturday	61.73	Thursday	72.68
Tuesday	77.41	Monday	72.78	Wednesday	63.29	Wednesday	74.60
Wednesday	77.62	Thursday	73.57	Friday	67.31	Tuesday	76.74
Thursday	77.62	Friday	75.46	Thursday	67.32	Friday	99.65
Friday	80.00	Wednesday	75.58	Tuesday	68.65	Saturday	109.48
Monday	84.40	Tuesday	75.94	Monday	69.42	Sunday	110.73
Sunday	85.27	Sunday	77.01	Sunday	81.04	Monday	135.52

TABLE 13

STANDARD DEVIATION OF HIGHWAY TRAFFIC VOLUME, FOR HOURS, FOR THE YEAR 1936-37, BY MONTH, IN CHRONOLOGICAL ORDER

Month	Michigan highway survey station numbers 650-655								
	650 East	650 South	650 West	651 North	651 East	652 South	653 South	654 West	655 North
	Number of vehicles								
January....	16	8	12	16	11	32	10	51	42
February...	15	8	11	11	9	27	9	33	26
March.....	16	9	11	16	12	35	12	53	65
April.....	21	10	16	21	15	45	12	52	81
May.....	31	14	25	26	19	74	12	67	223
June.....	36	14	31	24	19	58	10	60	166
July.....	53	20	46	34	27	78	11	74	335
August.....	48	18	40	32	24	91	12	98	307
September..	38	16	31	33	27	83	12	79	328
October.....	29	14	22	29	24	56	12	65	129
November..	22	11	18	35	27	55	11	63	78
December...	20	10	16	24	15	43	10	58	59

TABLE 14

STANDARD DEVIATION OF HIGHWAY TRAFFIC VOLUME, FOR HOURS, FOR THE YEAR 1936-37, BY MONTH, IN ASCENDING ORDER

Month	Michigan highway survey station numbers 650-651								
	650 East	Month	650 South	Month	650 West	Month	651 North	Month	651 East
	Number of vehicles								
February...	15	January...	8	February..	11	February..	11	February..	9
January....	16	February..	8	March.....	11	January...	16	January...	11
March.....	16	March.....	9	January...	12	March.....	16	March.....	12
December...	20	April.....	10	April.....	16	April.....	21	April.....	15
April.....	21	December..	10	December..	16	June.....	24	December..	15
November..	22	November.	11	November.	18	December..	24	May.....	19
October....	29	May.....	14	October...	22	May.....	26	June.....	19
May.....	31	June.....	14	May.....	25	October...	29	August....	24
June.....	36	October...	14	June.....	31	August....	32	October...	24
September..	38	September.	16	September.	31	September.	33	July.....	27
August.....	48	August....	18	August....	40	July.....	34	September.	27
July.....	53	July.....	20	July.....	46	November.	35	November.	27

TABLE 15

STANDARD DEVIATION OF HIGHWAY TRAFFIC VOLUME, FOR HOURS, FOR THE YEAR 1936-37, BY MONTH, IN ASCENDING ORDER

Michigan highway survey station numbers 652-655

Month	652 South	Month	653 South	Month	654 West	Month	655 North
	Number of vehicles						
February	27	February	9	February	33	February	26
January	32	January	10	January	51	January	42
March	35	June	10	April	52	December	59
April	45	December	10	March	53	March	65
December	43	July	11	December	58	November	78
November	55	November	11	June	60	April	81
October	56	March	12	November	63	May	223
June	58	April	12	October	65	October	129
May	74	May	12	May	67	June	166
July	78	August	12	July	74	August	307
September	83	September	12	September	79	September	328
August	91	October	12	August	98	July	335

TABLE 16

COEFFICIENT OF VARIATION OF HIGHWAY TRAFFIC VOLUME, FOR HOURS, FOR THE YEAR 1936-37, BY MONTH, IN CHRONOLOGICAL ORDER

Michigan highway survey station numbers 650-655

Month	650 East	650 South	650 West	651 North	651 East	652 South	653 South	654 West	655 North
	Percent that standard deviation is of mean of each hour of day								
January	73.30	72.33	77.18	88.13	87.18	76.06	86.90	75.54	66.99
February	78.24	78.81	82.14	93.54	89.00	87.71	97.08	66.58	52.97
March	70.74	71.31	73.70	83.54	73.67	71.91	76.81	68.06	66.42
April	76.23	71.02	82.60	79.75	75.14	69.26	75.07	64.23	69.55
May	79.23	72.79	86.15	75.45	74.06	82.44	74.25	68.75	109.28
June	74.72	65.86	83.31	71.52	70.02	62.94	69.18	57.70	77.63
July	74.77	75.09	80.11	71.33	68.25	63.49	67.34	59.70	88.09
August	68.26	64.81	71.69	69.36	67.53	65.10	69.00	65.45	77.11
September	74.12	66.89	80.52	78.63	80.21	72.88	72.46	68.28	113.75
October	74.62	70.15	77.88	75.79	73.33	66.13	74.39	65.07	74.87
November	66.83	65.01	70.76	82.23	81.25	67.67	75.84	67.29	61.50
December	68.97	68.93	72.74	87.85	77.15	72.04	75.39	66.44	60.47

TABLE 17

COEFFICIENT OF VARIATION OF HIGHWAY TRAFFIC VOLUME, FOR HOURS, FOR THE YEAR 1936-37, BY MONTH, IN ASCENDING ORDER

Michigan highway survey station numbers 650-651									
Month	650 East	Month	650 South	Month	650 West	Month	651 North	Month	651 East
	Percent that standard deviation is of mean of each hour of day, by months								
November..	66.83	August...	64.81	November.	70.76	August...	69.36	August...	67.53
August.....	68.26	November.	65.01	August...	71.69	July.....	71.33	July.....	68.25
December...	68.97	June.....	65.86	December.	72.74	June.....	71.52	June.....	70.02
March.....	70.74	September	66.89	March....	73.70	May.....	75.45	October...	73.33
January....	73.30	December.	68.93	January...	77.18	October...	75.79	March....	73.67
September..	74.12	October...	70.15	October...	77.88	September	78.63	May.....	74.06
October....	74.62	April.....	71.02	July.....	80.11	April.....	79.75	April.....	75.14
June.....	74.72	March....	71.31	September	80.52	November	82.23	December.	77.15
July.....	74.77	January...	72.33	February..	82.14	March....	83.54	September	80.21
April.....	76.23	May.....	72.79	April.....	82.60	December.	87.85	November.	81.25
February...	78.24	July.....	75.09	June.....	83.31	January...	88.13	January...	87.18
May.....	79.23	February..	78.81	May.....	86.15	February..	93.54	February..	89.00

TABLE 18

COEFFICIENT OF VARIATION OF HIGHWAY TRAFFIC VOLUME, FOR HOURS, FOR THE YEAR 1936-37, BY MONTH, IN ASCENDING ORDER

Michigan highway survey station numbers 652-655							
Month	652 South	Month	653 South	Month	654 West	Month	655 North
	Percent that standard deviation is of mean of each hour of day, by months						
June.....	62.94	July.....	67.34	June.....	57.70	February....	77.63
July.....	63.49	August.....	69.00	July.....	59.70	December...	60.47
August.....	65.10	June.....	69.18	April.....	64.23	November...	61.50
October.....	66.13	September....	72.46	October.....	65.07	March.....	66.42
November.....	67.67	May.....	74.25	August.....	65.45	January.....	66.99
April.....	69.26	October.....	74.39	December....	66.44	April.....	69.55
March.....	71.91	April.....	75.07	February.....	66.58	October.....	74.87
December.....	72.04	December....	75.39	November....	67.29	August.....	77.11
September.....	72.88	November....	75.84	March.....	68.06	July.....	88.09
January.....	76.06	March.....	76.81	September....	68.28	May.....	109.28
May.....	82.44	January.....	86.90	May.....	68.75	September...	113.75
February.....	87.71	February.....	97.08	January.....	75.54	June.....	165.70

scatter for the six weekdays is almost uniform and that that for Sunday is only moderately greater. The scatter for Sunday (as for each of the other days), however, is measured from its own mean. If it were measured from the mean of all

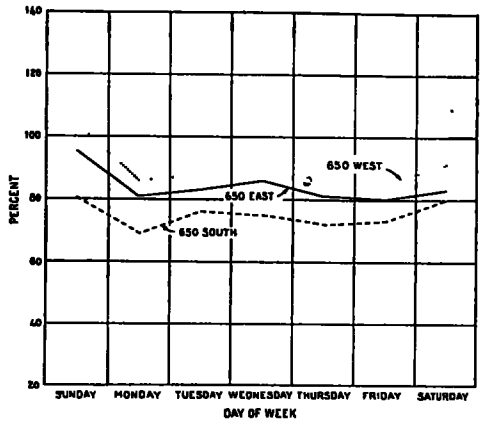


Figure 4. Coefficient of Variation of Highway Traffic Volume, by Days of Week, for Year 1936-37.

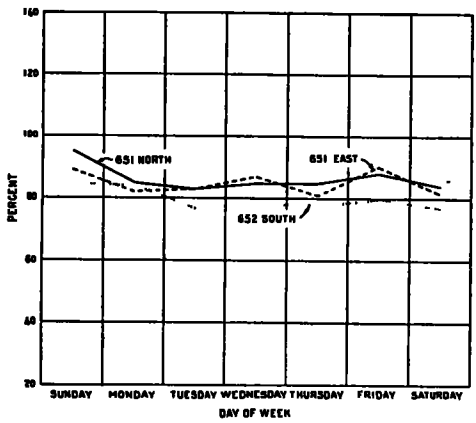


Figure 5. Coefficient of Variation of Highway Traffic Volume, by Days of Week, for Year 1936-37.

hours, the dispersion would be slightly greater for Sunday than shown. It is clear that for eight of the stations the gain in precision of estimate due to the elimination of Saturday is very small, and that due to the elimination of Sunday is moderate. The variation is important, however, for station 655 North, not only for Sunday and Saturday but also for Monday and Friday, two other days affected by the week-end scatter (Figs. 5-6). This tendency toward increased relative scatter for Friday to Monday in summer and week-end resort areas indicates that it may be desirable to separate the week-end days from the three midweek days for resort areas (Fig. 6, curve 655 North). For the other eight stations, however, the increased dispersion for Saturday and Sunday is not so great (Figs. 4-6).

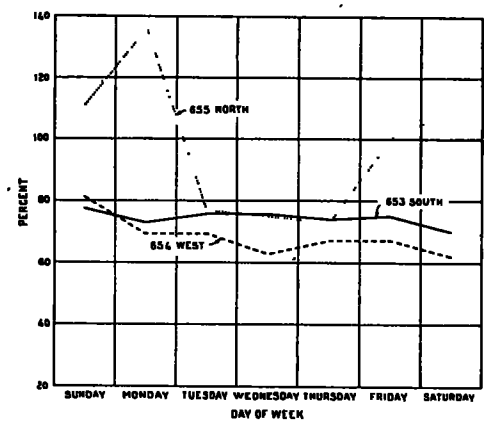


Figure 6. Coefficient of Variation of Highway Traffic Volume, by Days of Week, for Year 1936-37.

DISPERSION BY MONTHS

While the volume of traffic varies greatly between the winter and the summer months, the relative dispersion does not vary greatly. This is shown clearly in Figures 7-9 with the single exception of station 655 North. Even for this resort area, the irregularity is confined to the holiday months of May (Memorial Day) and September (Labor Day).

The relative constancy of dispersion for the months indicates that estimates for the annual mean can be made with fair precision provided an adequate

sample be taken from the constant day period and a proper pattern for the month is available for converting the monthly estimate to the annual monthly

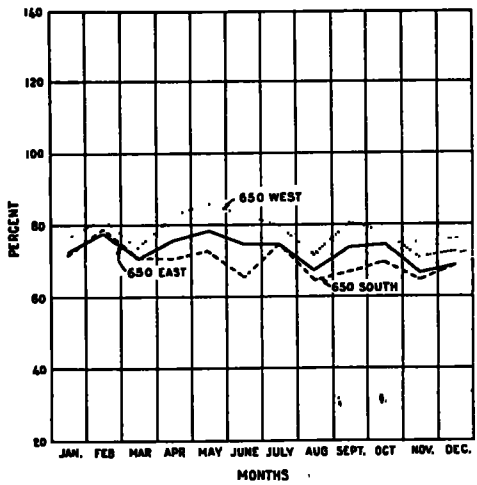


Figure 7. Coefficient of Variation of Highway Traffic Volume, by Months, for Year 1936-37

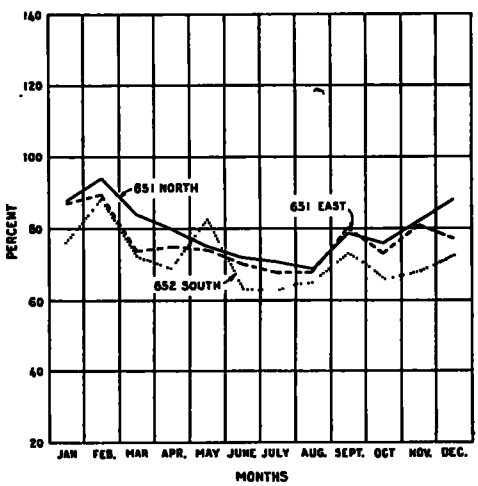


Figure 8. Coefficient of Variation of Highway Traffic Volume, by Months, for Year 1936-37

mean. An extension of this tendency is evident from the finding in the paper submitted to this Board a year ago; namely, that if the hours of the day be separated by months, the dispersion is limited to the function of scatter of the month.

Although the months of June, July, and August show smaller relative dispersion in hourly volume of traffic than the other months for the nine stations in Figures 7-9, it is clear that, in general, all the months may be used for estimating the traffic volume. The winter months show a tendency toward wide dispersion, and May and September reflect the influence of holidays in increas-

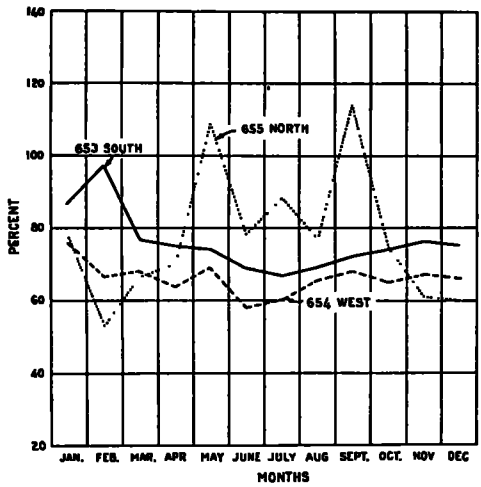


Figure 9. Coefficient of Variation of Highway Traffic Volume, by Months for Year 1936-37

ing dispersion. The departures for most stations, however, are not great enough to eliminate even these months from the sampling periods (Figs. 7-9). For station 655 North the dispersion is large for May and September, and for all months from May to September, inclusive, due to the summer vacation and week-end resort traffic. May and September would be the worst months for estimating the traffic at this point, and June and August the most dependable months. It would probably be advisable to include one month from each of the seasons among those of smallest dispersion for estimating traffic on this exceptional route.

A CASE HISTORY STUDY OF FATAL ACCIDENTS

By Earl Allgaier, *Special Investigator*

Highway Research Board and American Automobile Association

FOREWORD AND SYNOPSIS

This is a condensed report of a cooperative project of the Public Roads Administration, the Highway Research Board, and the American Automobile Association.

In 1936 and 1937 pursuant to an act of Congress the Highway Research Board cooperated with the then U. S. Bureau of Public Roads in an Investigation of Motor-Vehicle Traffic Conditions in the United States. As a part of this work the case histories of 1715 fatal traffic accidents were collected under the direction of Professor C. J. Tilden of Yale University. These accidents were studied and a report made thereon as Part 5 of the report to Congress published in six parts as House Document No. 462.

Feeling that this mine of valuable material had not been exhaustively explored the Public Roads Administration, and the Highway Research Board in 1939 arranged a cooperative project with the American Automobile Association whereby Mr. Earl Allgaier of the Association was assigned to make an independent study of the 1715 fatal traffic accidents occurring during 1936 and reported by eight States and eight cities.

The information was coded and sorting machines were used to obtain a large number of interrelationships of the various factors involved. The results consist of a large number of facts which will be useful in accident prevention programs but which cannot be summarized effectively in a few broad generalizations.

While the case histories studied were more complete than the usual accident report they indicated the lack of information concerning the circumstances which led up to the accident. This indicates the need for a shift in emphasis in accident reporting from a description of the collision and its effects to a more comprehensive analysis of the underlying circumstances that led up to the accident. A few of the more interesting points brought out by the study are as follows:

The median age of the pedestrians killed varied from a high of 65 from 5 to 6 a.m. to a low of 7 during the afternoon. When an index of the pedestrian death rate is computed on a population basis the age group 15 to 19 has the best record with an index of 0.19 while the group 75 to 79 has the worst record with 6.10. The index for older persons was found to be lower or higher but the number of cases was limited. Compared to all causes of death, traffic is most serious for pedestrians of 5 to 9 with an index of 11.76. It is less serious than the average for persons under 5 years of age and for those 60 or over.

Where data were available 54.3 per cent of the vehicles were 4 years old or less. Only 55.1 per cent of all vehicles were driven by the owner. Friends of the owner drove 18.0 per cent and employees of the owner 18.3 per cent of the cars. The median age of all drivers was 30. The median age of the driver when an owner was 36.5, when an employee 30.0 and when a friend of the owner 26.6 years. One-tenth of the drivers were 50 or over.

When an index of the estimated mileage driven per fatal accident for each age group is computed it indicates that those 19 and under had the poorest record with an index of 0.29 while those 50 to 59 had the best record with an index of 1.67. The probability of a driver being killed increases materially with age. Only 10.7 per cent of those under 20 were killed while 38.1 per cent of those 70 and over died as a result of the crash.

Drinking was reported for 11.6 per cent of the operators of passenger cars and 4.1 per cent of the operators of commercial vehicles.

An analysis of the case histories of 1715 fatal traffic accidents which occurred in 1936 was made to study the relative importance of the various factors involved and their relationship to each other. Each accident report not only had

more information than appears in the tabulations usually made of motor vehicle accidents but in addition the number of accidents was sufficiently large to be analyzed by quantitative methods. A total of 1455 case histories was obtained from the official records of the states of Connecticut, Massachusetts, New York, Rhode Island, New Jersey, Pennsylvania, Indiana and Michigan. The cities furnishing 260 additional case histories were Chicago, Ill.; Cleveland, Ohio; Detroit, Mich.; Evanston, Ill.; Louisville, Ky.; Milwaukee, Wisc.; and South Bend, Ind.

While it is true that the accidents were selected from states with the most complete records, it is not believed that the nature of the accidents will differ materially from those of the country as a whole. While the nature of the accidents is probably typical, the action that the state takes is probably not typical of the entire country. States with more complete accident reporting systems, driver examinations and licenses are more likely to take an active part once the accident has occurred.

The analysis of any motor vehicle accident is necessarily complex, not only because of the many factors involved but because of the many which are only partially known if known at all. The relationship of any one factor to any other is difficult to detect when there are many varying factors involved. In many cases differences have been obtained which are statistically significant but not large enough to have much practical use. In other cases differences have been indicated but only through the accumulation of parallel data will it be possible to state, with a fair degree of certainty, that the differences are significant.

The analyses of the various factors involved are necessarily based on the best judgment of the writer in interpreting, from the description of each accident, the relative importance of these various factors. So much is not known about these accidents, even though the information was carefully gathered that any statement relative to the cause or causes of an accident must be subject to rather large probable errors. While in many cases the physical conditions surrounding an accident such as type of highway, condition of roadway or lighting can be quantitatively measured with a fair degree of accuracy, many other factors, probably more important, cannot be so measured. These include such items as the condition of the driver, the attention of the driver to driving, the speed of the vehicle, the exact location of the pedestrian and many of the conditions which more indirectly led up to the accident. It is apparent that most reports describe the nature of the accident but fail to give completely the circumstances which led up to the situation. This is probably due mainly to the fact that those involved in the accident usually do not know just what those circumstances were. If the circumstances had been known, the accident could probably have been avoided. The very nature of an accident makes a clear-cut description of it practically impossible.

For the most worthwhile interpretation of the data obtained, a great deal more information is needed regarding exposure. It appears that many of the differences and trends obtained might be explained to a rather large extent by exposure. For instance, much is known concerning the number of drivers of various age groups involved in accidents. However, the true significance of this cannot be stated until the annual mileage of each age group is known. Similarly, any comparison of the number of pedestrians killed at rural and at urban intersections has little meaning until the relative amount of pedestrians and vehicular traffic is taken into consideration for the two places. The number of pedestrians killed in each age group will have much more meaning when the amount of walking on the streets is known for each group.

MOST SIGNIFICANT FINDINGS

While the study yielded a good deal of new information some of the findings were of especial significance. The more important of these are:

1. The median age of drivers involved in nighttime accidents was 4 years less than that of those involved in daytime accidents.
2. The percentage of drivers reported to be drinking was 3½ times as great for accidents at night as it was for accidents occurring during the day.
3. The greatest concentration of accidents was from 4 to 10 p.m., during which hours three-eighths of all fatal accidents occurred.
4. The median age of pedestrians killed varied from a high of 65 between 5 and 6 a.m. to a low of 7 for the hours from 1 to 2 and 3 to 4 in the afternoon.
5. While only one-tenth of the drivers were over 50, half of the pedestrians killed fell in this age group.
6. Only 55.1 per cent of the vehicles were driven by the owner at the time of the crash. Friends of the owner, not including relatives, drove 19.7 per cent of the passenger cars.
7. Based on estimates of mileage driven by various age groups, the group from 50 to 60 drove more miles per fatal accident than any other group, while the record was worst for the drivers under 20.
8. Older drivers involved in fatal accidents were more often killed than younger drivers. One driver out of ten in the age group under 20 was killed while four drivers out of ten in the age group above 70 were killed.
9. Drivers who had been drinking had a worse previous accident record than those who had not been drinking at the time of the accident.
10. Compared to all causes of death, traffic was more serious for pedestrians 5 to 9 than for any other age group.

SUMMARY OF FACTUAL DATA

The data do not lend themselves to many sweeping generalizations about the accident situation as a whole. Instead, this study substantiates a number of facts previously suspected and brings to light a number of others. Accident prevention programs can be much more effective if they are developed on the basis of adequate information and are designed to attack known causes. With this thought in mind a rather complete summary is here given. While many of the points appear to be of minor importance they deserve consideration in any serious attempts to reduce the total number of accidents.

Classification of Fatalities: In 92.8 per cent of the accidents there was only one death per accident. The remaining 7.2 per cent of the accidents accounted for 15.5 per cent of the total number of fatalities. The persons killed were classified as follows:

	Per cent
Pedestrians	53.5
Passengers	27.2
Drivers of motor vehicles	16.5
Riders of bicycles	2.6
Drivers of horse-drawn vehicles	0.2

This sample of fatal accidents had a larger percentage (53.5 per cent) of pedestrians killed than the 40 per cent estimated by the National Safety Council for the entire country in 1936.[1]

TIME ACCIDENT OCCURRED

Seasonal: The monthly distribution of fatal accidents shows that February had

[1] Accident Facts, 1937, p. 33.

the least (5.7 per cent) and October (10.5 per cent) the most. The three months having the largest number of fatal accidents were August, September and October.

Of all the accidents studied 15.2 per cent came from city records. This percentage remained very constant for the various months of the year. This indicates that, in general, weather conditions and other factors which change with the seasons affect city traffic about as much as they do rural traffic in so far as the number of fatal accidents are concerned.

Practically three-fourths (75.2 per cent) of the fatal accidents studied occurred under clear atmospheric conditions. The most frequent adverse condition was that of rain, present in one-eighth (12.1 per cent) of all accidents.

There is some indication that pedestrians killed during clear weather (median age of 47) tend to be younger than pedestrians killed during rains, snow storms or cloudy weather (median ages of 57, 58 and 59, respectively).

There is some indication that women drivers become involved in relatively fewer fatal accidents in the winter time than during the summer. While 7.2 per cent of the drivers involved in fatal accidents from April to September, inclusive, were women, only 4.3 per cent of the drivers for the remaining months were women.

The ages of pedestrians killed varied from month to month during the year. The median age for all pedestrians killed was over 50 for October, November and December. For the remainder of the months it was between 40 and 50. In February, May, June, August and September over 20 per cent of the pedestrians killed were under 10 years of age, as compared with 16.6 per cent for the entire year. Here again it is quite probably that exposure for all age groups is not the same for the various months. During the summer when school is not in session a good many of the children are more likely to be on the streets.

Most of these fatal accidents occurred on Saturday (19.8 per cent) and on Sunday (18.0 per cent). Fewest accidents were reported for Tuesday (11.0 per cent). Both in percentage and in actual numbers there were fewer commercial vehicles involved on Saturday and Sunday than on any other day of the week despite large totals of fatal accidents for these two days. Apparently the large number of fatal accidents on Saturday and Sunday cannot be charged to commercial vehicles.

The ages of cars involved in accidents were not the same for the various days of the week. Of the cars involved in accidents on Wednesday 24.7 per cent were 6 years old or older while on Saturday 45.6 per cent were in this age group.

The percentage of the pedestrians involved who had been drinking or were intoxicated was highest for Saturday and Sunday.

On Sunday not quite one-tenth of the persons killed as pedestrians were under 10 years of age while on Wednesday one-fourth of the pedestrians were in this age group. There is no significant variation in the sex of the pedestrian from day to day. About one-fourth of all those killed were females.

Hourly Variation: The hour from 6 to 7 p.m. had the greatest number of accidents (8.7 per cent) while the least number occurred between 5 and 6 a.m. (0.9 per cent). Three-eighths of the accidents (37.6 per cent) occurred from 4 to 10 p.m.

On the average 15.9 per cent of the vehicles involved were commercial vehicles. A very significant departure from this was for the period from 6 to 9 a.m. when two out of five of the vehicles involved in fatal accidents were in the commercial group. No doubt the relative amount of commercial vehicle traffic during various hours was an important fac-

tor. It is also possible that at this period of the day the effects of fatigue may become more serious, especially in the cases of operators who have been driving for most of the night previous. Only 7.0 per cent of all vehicles between the hours of midnight and 3 a.m. were commercial vehicles.

From 3 to 6 a.m. the proportion of drivers who were 40 or over was less than for any other three hour period. It is probable that relatively less driving is done by the older persons at night and if

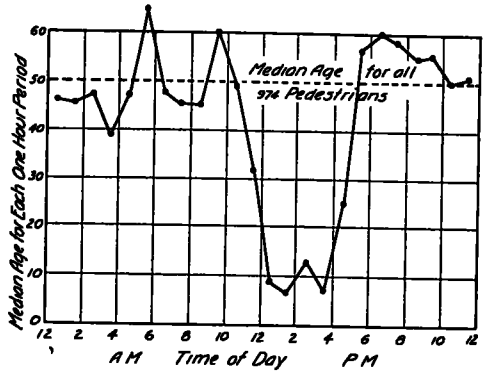


Figure 1. Age of pedestrian and hour accident occurred

the older people do drive, they compensate for their deficiencies. The period from 3 to 6 p.m. contains a higher percentage of drivers 40 or over than any other period.

The percentage of drivers with 5 years or less of experience involved in accidents from 9 to 12 p.m. was greater (26.4 per cent), and from 3 to 6 a.m. was less (12.7 per cent) than would be expected from the average throughout the day. The relative number of these younger drivers on the road is likely to account for a part of this difference.

Day and Night Accidents: Approximately one-third (34.0 per cent) of the accidents took place during clear daylight while an additional 7.0 per cent took place during cloudy daylight. Practically three out of five fatal accidents (59.0 per cent) took place during darkness or dusk. This agrees quite closely with the estimate made by the National Safety Council [1] that 60 per cent of the traffic fatalities for the country as a whole during 1936 occurred at night.

Drivers of 50 and over constituted a smaller proportion of those involved in nighttime accidents than of the drivers involved in daytime accidents. Two possible explanations of this are: (1) older drivers operate more cautiously at night and (2) older drivers do less driving at night. Many elderly persons say that they do not drive at night because they are bothered by glare or it is difficult for them to see.

While women constituted 9.0 per cent of the drivers involved in accidents in the daytime, they formed only 2.7 per cent of the drivers in fatal accidents at night. It is probably that women do a relatively greater proportion of their driving during the day. Though most of the fatal accidents occurred at night, three-fifths of the women drivers in them were involved between 6 a.m. and 6 p.m.

The data indicated that drivers involved in accidents at night had less experience than those involved in daytime accidents. While about one-fourth (25.6 per cent) of the drivers involved in daytime accidents had five years or less of experience, about one-third (32.5 per cent) of those involved in night accidents could be so classified.

Pedestrians Killed at Night: There was a decided variation in the age of the pedestrians killed for the various hours of the day. (See Fig. 1.) During the four hours from noon to 4 p.m. the median ages of the pedestrians killed were 9, 7, 13, and 7 respectively, as compared with the highest median of 65 for the hour from 5 to 6 a.m. Children are of course kept off streets during night hours to a large extent. The hours from noon to 5 p.m. appear to be most dangerous for child pedestrians under 10 years of age.

During each of these hours from three-eighths to five-eighths of the pedestrians killed were in this age group while for the whole day only one-sixth of the pedestrians were under ten years of age.

Of all the pedestrian deaths, 62.8 per cent occurred at night. While the median age of the pedestrians killed during the night was 55, of those killed during the day it was only 9. Apart from differences in exposure the fact that older persons have greater difficulty in seeing at night so that traffic hazards become relatively more serious to them may be an important factor.[2] An A.A.A. research report gives the average glare resistance score for persons 25 years of age as 18.6 and for those 65 years of age as 5.4. Not only are these persons slower in perceiving hazards but they are slower to react once they see the danger. This indicates the need for emphasizing to adults the dangers of walking at night and to children the hazards of traffic in the daytime.

LOCATION OF THE ACCIDENT

All except 4.2 per cent of the fatal accidents occurred in the four major locations—rural and urban, intersections and non-intersections. The percentages of accidents occurring at various locations were as follows:

	Per cent
Rural non-intersection	37.2
Rural intersection	5.6
Urban non-intersection	28.5
Urban intersection	24.5
Off the highway	0.3
Railroad crossing	1.3
Bridge	1.4
Underpass or overpass	0.6
Private driveway or alley	0.6

Drivers 50 and over seemed to have a disproportionate share of their fatal accidents at rural intersections. While about one-tenth of the drivers involved in fatal accidents were 50 or over, one-fifth of those involved in fatal accidents at rural intersections were in this age group.

Child pedestrians under 10 years of age were more frequently killed at urban non-intersections and less frequently at rural non-intersections than would be expected from the total number of pedestrians killed at the various locations.

Women pedestrians were killed relatively more frequently than men at rural and urban intersections and less frequently than men on rural highways between intersections. It seems probable that women walk less on rural highways than men, especially at night, and this would account for a part of the difference noted.

Traffic Control: In seven accidents out of eight (87.5 per cent) there was apparently no traffic control functioning. Traffic signals were present in 6.7 per cent of the accidents, stop signs in 3.1 per cent and railroad warnings signs in 1.3 per cent. Other means of control were present in lesser numbers.

Obstructions to Vision: Aside from parked cars few obstructions to vision were mentioned. The most frequent were obstructed windshields, pillars in roadway and snow banks.

ROAD SURFACE AND CONTOUR OF HIGHWAY

Surface Condition: Nearly three-fourths (72.1 per cent) of the accidents occurred on dry and clean surfaces. An appreciable number (17.9 per cent) occurred on wet surfaces and an additional 6.3 per cent on icy surfaces. In only one accident out of 25 was a defect in the road reported that might have contributed to the accident.

Grade and Alignment: Practically nine-tenths (89.3 per cent) of the accidents occurred on the level while most of the remainder occurred on a grade and less than 1 per cent on a hillcrest. One-eighth occurred on curves while the remainder occurred on the straight-away.

[2] Research Report No. 6 Safety and Traffic Engineering Department, American Automobile Association.

MOTOR VEHICLES INVOLVED IN THE ACCIDENT

Age and Condition of Vehicle: The 1715 fatal accidents studied involved 1626 passenger cars, 3 passenger cars with trailers, 370 commercial vehicles, 43 commercial vehicles with trailers, 22 taxis, 27 passenger busses, 1 school bus, 6 emergency vehicles, 38 motorcycles, 24 trains, 52 bicycles and 4 animal-drawn vehicles.

The median age of all motor vehicles involved (including commercial vehicles and motorcycles) was 4 years with 54.3 per cent being 4 years old or less. One vehicle out of 8 was 8 years old or older. The commercial vehicles median age was 2 years and 72.6 per cent were 4 years old or less.

The prevalence of reported defects in vehicles was about the same for both commercial and passenger vehicles.

Since in only 7.4 per cent of the accidents was there mention of the condition of the vehicle any conclusion regarding the effect of certain defects on accidents must be highly tentative. Of the 117 cases of vehicles stated to be defective, brakes were mentioned in 66 cases, tires 13, windshield or wiper 13, steering 9 and in 16 other cases lights with one defect or another.

Direction of Travel: Eighty-nine per cent of the vehicles involved were going straight ahead or following a curve in the highway, while 3.4 per cent were overtaking another vehicle, 2.4 per cent were stopped in traffic or on the roadway, and 3.3 per cent were making right or left turns. In a few other cases the vehicle was parked, backing, pulling from the curb, slowing down or making a U turn. In this connection it is quite significant to note that while 10.3 per cent of the drivers of vehicles going straight ahead had been drinking or were intoxicated, only 1.9 per cent of the drivers making turns were so classified.

Ownership: A surprisingly large number of vehicles involved in fatal accidents were not operated by the owner at the time of the accident. In fact only 55.1 per cent of the vehicles were operated by the owner at the time of the crash. 18.3 per cent were operated by employees of the owner, 18.0 per cent by friends of the owner, and 7.4 per cent by relatives of the owner, including wives. Since the ownership was given in 90 per cent of the cases, the figures are probably fairly typical of fatal accidents in general.

While 58.5 per cent of the passenger cars were operated by the owner only 21.6 per cent of the commercial vehicles were so operated. Friends of the owner operated 19.7 per cent of the passenger cars but only 5.9 per cent of the commercial vehicles. 59.4 per cent of the commercial vehicles were operated by employees of the owner.

Relatively, women were more often driving a car belonging to another member of the family or a friend than were men but they were less often driving the car of their employer. Women though constituting 5.7 per cent of the drivers, were drivers of 15.3 per cent of the cars owned by a member of the family, 8.9 per cent of the cars owned by friends, 5.2 per cent of the cars driven by the owner but no women were involved in accidents while driving a vehicle belonging to an employer.

Drivers involved in accidents while driving their own car had a median age of 36.5 years, those driving their employer's car 30.0 years and those driving cars belonging to friends 26.6 years. The experience of the driver followed somewhat these same trends. Of the drivers operating their own cars ¼ had 15 or more years of experience while only 13.7 per cent of the operators of friend's cars had that much experience.

One-fifth of all the drivers were reported to have had previous accident or violation records. Compared with this,

26.9 per cent of the drivers operating their employer's vehicles had previous records. This difference may be accounted for in part by the fact that employees of the owner probably do more driving than other drivers and so expose themselves to more accident situations.

THE DRIVERS INVOLVED

Age: Since the driver is probably the most important factor in the accident, a number of analyses were made to determine the relationship of various characteristics of the driver to the accident.

The median age of all drivers involved in fatal accidents was 30 years. One-tenth were over 50 years of age. The distribution of the drivers of various age groups follows very closely the distribution of drivers involved in fatal accidents in Connecticut for 1932 to 1936.[3] Drivers 20 and under had the most fatal accidents for the number registered while the drivers 46 to 50 had the least. On this basis the latter group had three times as good a record as the former.

When estimates of the miles driven by various age groups [4] are combined with the number of registered drivers in each group and the number involved in fatal accidents, the differences in the age groups is accentuated. (See Fig. 2.) With the drivers involved in the 1715 fatal accidents grouped by ten year periods it appears that the age group 50 to 59 drives the most miles per fatality while the group under 20 drives the least number of miles. The former group apparently drives nearly 5.7 times as far per fatal accident as does the latter group. While the data for drivers over 60 are more limited, they indicate that the miles driven per fatal accident decline consistently with age.

While 18.6 per cent of all drivers were operating commercial vehicles at the time of the accident only 8.7 per cent of the drivers 50 and over were in this group. It seems probable that organizations hiring drivers tend to discourage older persons from driving. On the other hand there is little pressure brought to bear on the individual who drives his own car even though his abilities have fallen off somewhat with age.

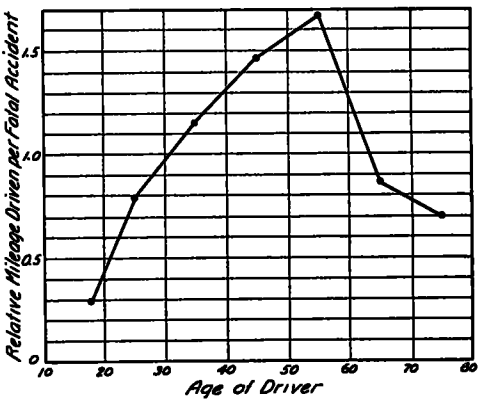


Figure 2. Miles driven per fatal accident. Mileage estimated from A.A.A. driver test records and number of drivers registered in Connecticut for 1932-36 for each age. Drivers involved in 1715 accidents in 1936.

As would be expected the older drivers involved in fatal accidents had more experience than the younger drivers. However, for drivers 40 and over the amount of experience did not appear to increase greatly with age. This indicates that the drivers in the higher age groups probably learned to drive at a much later age than the younger drivers.

The probability of the driver being killed in a fatal accident increases materially with age. (See Fig. 3.) Of all the drivers involved in fatal accidents 14.6 per cent died as a result of the crash. While 10.7 per cent of the drivers under

[3] "Motor Vehicle Traffic Conditions in the United States," Part 6, "The Accident Prone Driver," 75th Congress, Third Session, House Document 462, Part 6.

[4] "Drivers 20 to 40 Rate Highest on Tests," Earl Allgaier, *Psychological Bulletin*, Vol. 35, p. 651.

20 were killed the percentage of those over 70 who were killed was 38.1. Since there is no reason to assume that older drivers get into more violent accidents than do other drivers it is probable that the decreasing vigor which accompanies old age is an important factor. This points to the possibility that perhaps the high death rate among elderly pedestrians is due to their inability to recover from certain accidents that would not prove fatal to younger persons.

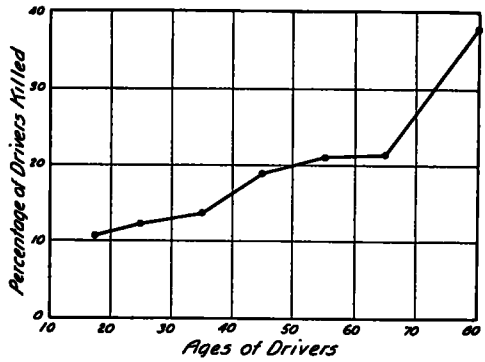


Figure 3. Likelihood of Drivers involved in fatal accidents being killed. 2044 Drivers involved. 299 killed.

Women Drivers: Only 5.7 per cent of the drivers involved in fatal accidents were women. While accurate figures on the mileage driven by men and women are not known, estimates can be made. In Connecticut, for example, about one-fifth of the registered drivers are women. Of the persons taking the AAA driver tests the men indicated about twice as much driving per year as the women. If these figures are approximately correct then one-tenth of the driving would be done by women and consequently we would expect them to have one-tenth of the total fatal accidents. It thus seems that exposure is not sufficient to account for the difference in the number of fatal accidents involving men and women drivers.

Drinking appears to be more prevalent among men drivers involved in fatal accidents than among women drivers. While 10.2 per cent of the men drivers were reported to have been drinking or were intoxicated, the figure for women was only 3.3 per cent.

Women appear more likely than men to get into collisions with fixed objects but less likely to hit pedestrians.

Experience: The median number of years of driving experience for drivers involved in fatal accidents was nine. Drivers having less than one year of experience constituted 4.9 per cent of the entire group; 11.8 per cent of the drivers had 20 years or more of experience. About the same percentage of drivers with little experience got into accidents on slippery surfaces as did drivers of considerable experience.

On the average the women involved in fatal accidents have had less experience than men. The median number of years of experience for women was seven while for men it was nine. Of all drivers with experience reported less than 5 per cent were women. Of the drivers with less than a year's experience 20.3 per cent were women. This indicates the necessity especially for women who are learning to drive to avoid the hazards of traffic as far as possible.

Occupation of Driver: In about half of the fatal accidents the occupation of the driver was given. Of these cases practically three drivers out of ten were chauffeurs or drove as their main business. An additional two out of ten were skilled or semi-skilled workers while common laborers constituted one out of ten involved in fatal accidents. Other occupational groups showed smaller percentages.

While 7.7 per cent of all the drivers had collisions with fixed objects this percentage was practically doubled for the following groups: (a) professional persons, (b) common laborers, (c) house-

wives and domestic workers, (d) youths, students and unemployed. On the opposite extreme are the drivers and chauffeurs. Only 2.1 per cent of them had collisions with fixed objects.

Previous Record of Driver: One driver out of five was reported to have had a previous accident or law violation. Half of these had more than one accident or violation. The records of 80 per cent of the drivers were not mentioned in the accident reports. It is likely that this latter group had a large number of drivers who had previous clear records. In cases where the coroner takes action he, either consciously or unconsciously, is more likely to hold a driver with a past record than a driver where no past record is reported. Of the drivers with recorded previous records 7.6 per cent were held while 4.6 per cent of the drivers with no past records were held. There was a record of the police arresting two drivers out of each five involved in fatal accidents. The past record of the driver did not seem to have any effect on the probability of being arrested or on the action of the court in punishing the driver. In contrast to this the motor vehicle departments were more apt to suspend the license of a driver with a past record than that of a driver with no past record. While 29.1 per cent of the drivers with no past records had their license suspended for 30 days or more, 43.8 per cent of the drivers with past records had their license so suspended.

There was a record of one out of ten of the drivers involved in fatal accidents having had some type of previous punishment such as fines, jail sentence or revocation of driver's license. Six-tenths of these drivers received fines only.

In most of the reports there was hardly information enough to weigh accurately the importance of the various factors which contributed to the accident. While it may generally be conceded that the driver is probably the most important factor in most accidents just which actions of the driver are most serious is far from a settled question. As far as could be determined from the reports about 17 per cent of the drivers were apparently not at fault, that is, they were driving in a normal fashion and in general following the rules of good driving. A number of these, however, might possibly have been able to avoid the accident by taking extreme measures. An example might be driving off the highway into the ditch to avoid a head-on collision with someone driving on the wrong side.

Speed Too Fast for Conditions: While it is possible in most accidents to assign a number of main and contributing causes, in this study not more than two were assigned to any one driver or vehicle. An analysis of these causes indicated that speed too great for the proper control of the car was the major cause of most fatal accidents. Of 2126 important causes listed 39.2 per cent included speed too fast for vehicles, pedestrians or objects in the path of the vehicle. Included here are such cases as a driver hitting a pedestrian at night because he did not see the pedestrian soon enough or hitting some object in the roadway because of poor visibility due to atmospheric conditions. In other words, the stopping distance of the car was in excess of the clear highway visible ahead.

In another category are those classes of excessive speed where normally the road would be clear but where the driver under normal conditions should expect occasionally other drivers or pedestrians to come into the pathway of the vehicle. These include chiefly going too fast through intersections so that it would be impossible to yield to another driver should he come from the side, going too fast past pillars, parked cars, etc., where one might expect pedestrians. This group included 21.5 per cent of the causes.

Still another category of excessive speed included accidents where the driver

was not able to control the car because of the condition of the highway or the car. Driving too fast for a slippery road or road alignment are included in this group as well as speed that is excessive for proper control even with the highway and car in good condition. These causes account for an additional 15.9 per cent of the total.

While these percentages may appear to be rather high for speed as a factor, it is assumed that the good driver will adjust his driving to the conditions he finds. It appears that the factor of safety used in driving is relatively small under normal conditions and even this small factor of safety is lost by a great many drivers when driving under adverse conditions This in turn results in accidents.

Vehicle Control: Aside from speed the next most important factor appeared to be improper control of the vehicle. This covers a multitude of operations for which there is no apparent reason and in some respects is not a cause but a part of the accident. Here are included such items as running off the road without any apparent reason, or driving on the wrong side. Inattention may be a large factor but rather than try to guess what the basic cause may be it seems better simply to state the fact as objectively observed. A man may drive off the road because he goes to sleep, because he was distracted by a passenger, because he had heart failure or possibly because something went wrong with the steering mechanism but unless one of these other factors is established it seems best to merely state the fact that he drove off the road for no apparent reason. Of the total there were 104 cases of driving on the wrong side of the road and yet there was no evidence to indicate the cause such as excessive speed, slippery road surface, etc. Improper control of the vehicle as thus defined accounted for 12.9 per cent of the causes.

Other Deficiencies: Other factors which seem to be clearly identified as causes of the accident are: inattention and distraction 3.2 per cent of the total causes listed, violation of traffic control 2.8 per cent, physical condition of the driver 2.2 per cent, known or suspected defects of the vehicle 1.9 per cent, and miscellaneous causes 0.4 per cent.

Accident Prevention Education: After the foregoing analysis of the causes of accidents as applied to the driver it appears that the greatest prospects for the reduction of accidents lie in the education of the driving public to proper speeds to be driven to maintain a proper margin of safety for all conditions of the highway, the car and other traffic including pedestrians. It appears for example that a great many accidents involving pedestrians after dark are due to the fact that the motorist does not see the pedestrian until it is too late to avoid a collision. The motorist fails to appreciate the limited distance he can see a pedestrian or else he is willing to take the chance that the roadway will be clear. In his particular instance education as to the difficulties in seeing at night and also of the possibilities of pedestrians being on the roadway should be effective in bringing speed down to a point where the stopping distance does not exceed the clear distance ahead. In case the driver still fails to drive with a reasonable factor of safety, regulation may be necessary. Structural changes in highways may make the relative danger apparent to the driver. This would call for a uniform use of danger or warning signs, scientific speed zoning, etc. It is not the danger itself that causes the trouble but the danger which the driver does not realize exists.

THE PEDESTRIAN

The importance of the pedestrian in the accident problem is indicated by the fact that over half (53.5 per cent) of the persons killed in the 1715 fatal accidents

were pedestrians. While the pedestrian does not have under his control a powerful machine capable of inflicting serious injury he nevertheless is expected to cooperate with the driver since both must use the same roadway at times and especially so since the pedestrian is the one from a personal point of view who is most likely to suffer in an accident.

Ages of Pedestrians Killed: It is quite evident that traffic deaths among older pedestrians is far in excess of what would be expected from the number living in each age group. (Fig. 4.) In the present

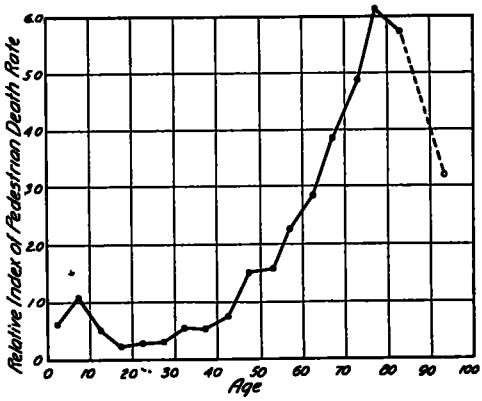


Figure 4. Pedestrian Death rate by Age groups. 983 pedestrians

study approximately half (50.4 per cent) of the pedestrians killed were 50 years or older. Compared with this only 17.1 per cent of the U. S. population is 50 or over. This confirms other studies that the death rate among elderly pedestrians is much higher than that among younger pedestrians. For the construction of a table of index figures for the death rate of pedestrians of various ages a value of unity has been assumed for the average of all age groups. On this basis the 5 year age group from 15 to 19 inclusive had the best rate of 0.19 while the group from 75 to 79 had the highest rate of 6.10. In other words, the chances of a person in the latter age group being killed as a pedestrian are 31.4 times as great as that of a person in the younger age group. The age group 5 to 9 and all persons 45 and over had a higher death rate than the average.

If the pedestrian death rate followed the general death rate due to all causes, the two curves would follow each other. However, although 1 per cent of the persons dying from all causes are between 5 and 10 years of age nearly 11 per cent

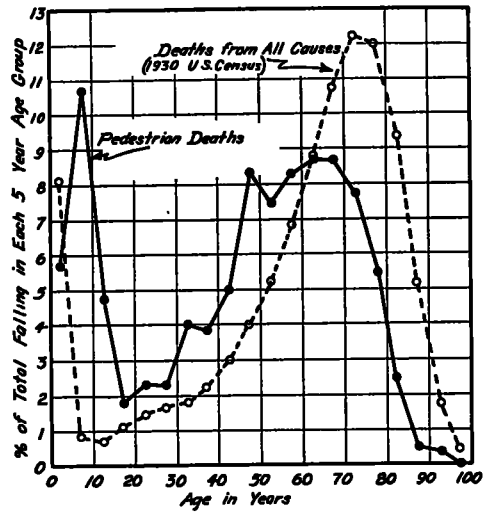


Figure 5. Pedestrian and all deaths by ages

of all pedestrians killed were in this particular age group, showing that relatively, traffic is a very important cause of death for children 5 to 10 years of age. This comparison is shown graphically for all ages in Figure 5. It is interesting to note from the graphs that it is the persons under 5 and over 60 who have a pedestrian death rate lower than would be expected from the general death rate, while those between 5 and 60 have a higher pedestrian death rate than would be expected.

Another interesting analysis is shown in Figure 6. On the basis of life expectancy tables the total number of years of expected life lost by pedestrians killed

in each age group was calculated. As shown in Figure 6 this is greatest for the group of 5 to 10. Nearly one-fourth of all the expected life lost comes from this group.

Female pedestrians killed constituted a larger proportion of the age group from 0 to 9 than of any other age group. This may indicate a tendency for boys of this age group to adapt themselves more readily to the hazards of traffic than girls. It would not be expected that in this group exposure is relatively greater for the girls.

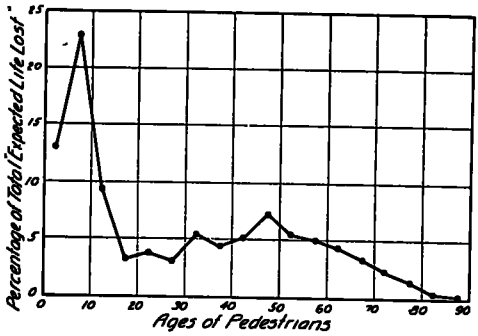


Figure 6. Years of expected life lost by pedestrian deaths at each age. 983 pedestrian deaths. Expected life from U. S. Census.

Pedestrian Accident Factors Unknown: In most fatal pedestrian accidents the story of the accident must come from the driver. Usually the driver, even if honest, does not have a clear picture of what happened or he might have avoided the accident. Because of this the important factors relative to the pedestrian are not known. Frequently only the point of contact is known instead of the more important questions as to where the pedestrian came from, where he was going, how fast he was traveling and how much of a chance an alert motorist would have had of seeing him. Careful interviews of pedestrians who have been hit but survived might yield clearer answers to these problems than are possible from study of fatal accidents only.

Nearly six-tenths (58.2 per cent) of the pedestrians were crossing the roadway when hit. The large majority of these were crossing a city street, with twice as many killed crossing between intersections as at intersections.

Of all pedestrians killed 30.2 per cent were walking in the roadway. The action of half of these was not known. Of those walking in traffic two were killed walking against traffic for each three killed walking with traffic. Other important actions involving pedestrians in the roadway included playing, working on a parked car and hitching rides.

INTOXICATION

Drinking appears to be more of a factor in night than in day accidents. While 4.1 per cent of the drivers involved in day accidents were alleged to have been drinking or were intoxicated, 14.5 per cent of those involved in night accidents were so reported. This may partially explain why, on a mileage basis, the accident rate is so high at night. The night rate may not be due entirely to darkness as such but to the factors that are associated with darkness—more drinking being one of them.

The prevalence of drinking as far as these fatal accidents were concerned reached a peak from 12 to 3 a.m. when one-fourth of the drivers were reported to have been drinking or were intoxicated. Drinking during the daytime seemed to be much less of a problem as only 4.3 per cent of the drivers involved in accidents from 6 a.m. to 6 p.m. were so reported. While no doubt many cases of drinking were not reported the general trend for the various hours of the day is probably substantially correct.

Drinking among pedestrians appeared to be most frequent from 9 p.m. to 6 a.m. when one-fourth of pedestrians killed were reported to have been drinking or were intoxicated, while between 6 a.m.

and 6 p.m. only 3.2 per cent of the pedestrians were so reported. These figures indicate the necessity of keeping drinking pedestrians off the street during certain hours of the night and also the probability that alcohol is an important factor in the production of night pedestrian accidents.

Drivers who were intoxicated or who had been drinking were in a disproportionate number of accidents on curves. While 8.3 per cent of the drivers involved in accidents on the straightaway were reported to have been drinking or were intoxicated, this group constituted 19.6 per cent of the drivers involved in accidents on curves. It seems that the difficulty drinking drivers have in controlling a car on the straightaway is materially increased on curves.

While nearly one-tenth of all drivers were reported to have been drinking or to be intoxicated, only 2.8 per cent of the drivers of cars belonging to an employer fell into this group.

Drinking among pedestrians between 20 and 59 is much more prevalent than among those older or younger; 23.6 per cent of this group had been drinking or were intoxicated while only 8.6 per cent of those 60 or above could be so classified. There was no record of any pedestrians under 20 having been drinking.

While the condition of the pedestrian was given in one-fifth of the cases it is quite apparent that drinking was the most common condition that might have contributed to the accident. Inattention, confusion and physical defects such as hearing and vision, were mentioned in a number of cases.

BICYCLE ACCIDENTS

Of the 52 bicycle accidents studied, 71 per cent took place during the 5 months from May to September inclusive. Three-fourths of the accidents took place between 3 and 12 p.m. and were fairly evenly distributed over that period. Three accidents out of five took place during hours of daylight. Only 8 per cent of the accidents occurred during adverse weather conditions. While 27.3 per cent of all fatal traffic accidents occurred on slippery surfaces (ice, snow or wet) only 15 per cent of the bicycle accidents occurred on such surfaces. In nearly all cases the bicycle was going straight ahead. In three cases the bicyclist was overtaking a car and in one case making a left turn.

Most of the bicycle riders were under 20 years of age; a few were over 40; there was only one between 20 and 39 years of age.

SUGGESTED PROBLEMS FOR RESEARCH

Even though this detailed analysis of 1715 fatal traffic accidents has yielded further information, there is still much to be learned about the nature of traffic accidents. Much of the time and money spent on the gathering of accident statistics which now only serve to confirm what is already known and accepted might be used more profitably in special studies of factors on which information is sadly lacking. We are particularly deficient in data on accident exposure under various circumstances. For the guidance of those interested in pursuing further some phases of traffic accidents several suggestions for further research are offered:

1. Interviews with pedestrians who have been hit, but survived in order to obtain more information as to what they did to become involved in the accident and what they might have done to avoid it.
2. The miles driven per year for vehicles of various ages. Where inspection is required taking speedometer readings at successive inspections would give a good deal of information. (Washington, D. C. asks for this information when license plates are issued.)

3. The prevalence of defects among vehicles of various ages.
4. The number of vehicles of each age registered. An analysis of registration would yield this information.
5. The relationship of the age of the car driven to the age of the driver.
6. The prevalence of defects among cars involved in accidents and other cars.
7. The relative number of miles driven by commercial and non-commercial vehicles for various days of the week and for various hours of the day. A count of traffic on the highway for various locations would give an approximate figure.
8. The relative number of vehicle miles for various hours of the day, various days of the week and various months of the year.
9. The relative amount of driving done by friends of the owner, relatives of the owner, employees of the owner and the owner of the car itself.
10. The amount of driving done by men and women during the day and during the night. A count of traffic at several points would give some indication of the relative amount of driving done by the sexes.
11. The amount of driving done by the various age groups during the various hours of the day.
12. The partial correlation of accident records with age and experience of the driver to determine if it is age or experience which makes the record of young drivers so poor.
13. The percentage of operators overdriving their headlights for various age groups.
14. The percentages of the time various weather conditions prevail such as snow, rain, clear, cloudy, etc.
15. The relative number of pedestrians crossing the street at the various locations as in the crosswalk, in the middle of the block, with the green signal, etc.
16. Additional data on the relative percentage of pedestrians and drivers who have been drinking for the various hours of the day.
17. The prevalence of the various types of unsound walking practices among pedestrians of various age groups.
18. The number of persons riding in cars involved in fatal accidents to determine if the greater likelihood of a passenger being killed than a driver is due to the greater number of passengers involved.

DEPARTMENT OF SOILS INVESTIGATIONS

C. A. HOGENTOGLER, *Chairman*

PRESENT STATUS OF SOILS INVESTIGATIONS

BY C. A. HOGENTOGLER, *Senior Highway Engineer*

AND E. S. BARBER, *Junior Highway Engineer*

Public Roads Administration

SYNOPSIS

A review of soil investigations during the past decade discloses that notable progress has been made in the use of indicator and compaction tests by the various State highway laboratories and the utilization of test data in subgrade treatment and in the design and construction of road surfaces, base courses and embankments. Use of consolidation and shear tests has lagged due to the fact that the correlation of test results with performance of soil structures and foundations has not been forthcoming. Needed especially in this respect are observations of settlements of bridge piers, sway or creep of retaining walls and abutments and of the stability of embankments. Yet, procedures of test and applications of test data for determining settlements have attained a more or less standard status and are particularly applicable for estimating settlements of embankments due to consolidation of the foundation soil because the relatively large size of the loaded area under the embankment approaches more nearly the conditions of test than is met in the smaller loaded areas under footings and bridge piers. By use of such estimates and also those furnished by the application of shear test data in theories now available as yard sticks, every highway embankment becomes a potential source of the information needed in the correlation necessary to bring the consolidation and shear tests to a practical status.

The past decade saw considerable effort devoted to investigations of the engineering properties of soil, and a commensurate amount of material published on the results attained. The findings applicable to highway construction were compiled under the title "Soil Stabilization and Soil Mechanics" and published as Part II, Proceedings, Highway Research Board, 1938 (1).[1] Included was a discussion of four general groups of tests as follows:

1. Indicator tests.
2. Compaction tests.
3. Consolidation tests.
4. Shear tests.

It seems appropriate at this time to review the status of these groups with respect to (a) apparatus and testing procedures; and (b) the utilization of the test data in practice.

[1] Numbers in parentheses refer to list of references at end.

Indicator Tests.

The determinations which indicate rather than disclose quantitatively the engineering properties of soil are:

Mechanical analysis.
Liquid limit.
Plastic limit.
Plasticity index.
Shrinkage limit.
Centrifuge moisture equivalent.
Field moisture equivalent.
Shrinkage factors.

The significance of these determinations for disclosing the properties of subgrade soils, and their use in a classification of subgrades based upon the performance of the soil was first discussed in *Public Roads,* May 1929 (2). Procedures for making the determinations were published in *Public Roads,* October 1931 (3).

By 1933, more than 30 State highway departments had included at least several

of these determinations in their routine laboratory examinations of road materials. Procedures for making all the indicator determinations were adopted as standard methods by the American Association of State Highway Officials in 1934 (4); as tentative standard procedures by the American Society for Testing Materials in 1935 (5); and were advanced to standard status by the latter Society in 1939.

Specifications for stabilized road surfaces, and for stabilized base courses based on properties of the materials disclosed by the mechanical analysis, the liquid limit and the plasticity index were adopted as standards by the American Association of State Highway Officials in 1938 (4); and are now being considered as tentative standards by the American Society for Testing Materials.

Compaction Tests.

Tests which disclose the moisture content required by soil to attain maximum density at a given compactive effort produced by tamping, were described by R. R. Proctor in the Engineering News-Record in 1933 (6) for use in the construction of earth dams. Compaction tests employed by the California Highway Department and which utilize static pressures in the determination of moisture content-density relations of compacted soil for use in highway embankments were described by T. E. Stanton in 1938 (7).

Use of the Proctor tests generally in highway construction, and especially to determine the effect of chemical, portland cement and bituminous admixtures for changing the density and stability of soil was discussed in *Public Roads* in February 1935 (8), and May 1936 (9).

Procedures for performing the compaction tests on soils were adopted as standard by the American Association of State Highway Officials in 1938 (4), and are now being considered as tentative standard by the American Society for Testing Materials.

Specifications for the selection of embankment materials, based upon compaction test data were adopted as standard by the American Association of State Highway Officials in 1938.

Use of the compaction test data in the design and construction of soil-cement road bases was described in the Proceedings, Highway Research Board, 1937 (10). Procedures for performing the compaction tests on soil-cement road mixtures have received consideration as tentative standard by the American Society for Testing Materials and have been published for information by that Society in the proceedings of the 1939 annual meeting.

In the Proceedings of the Highway Research Board, 1938, Vol. 18, Part II, 24 State highway departments reported use of the compaction tests.

Consolidation and Shear Tests.

The consolidation tests were described by Charles Terzaghi in the Engineering News-Record in 1925 (11), and in *Public Roads* in 1927 (12). A complete exposition of the theory of consolidation, the consolidation tests and their practical application was published in *Public Roads* in 1937 (13).

As yet, no effort has been made to standardize the consolidation tests, and according to the Proceedings of the Highway Research Board referred to above, only nine State highway departments have obtained apparatus for making the tests.

Shear tests for soil have been described repeatedly since the time of Coulomb. The various types are described in the Compendium on Soil Testing Apparatus, Proceedings of the Eighteenth Annual Meeting, Highway Research Board, Part II (1).

As in the case of the consolidation tests, no effort to standardize shear tests has

yet been made; and, according to the Research Board Proceedings (1) only nine State highway departments are equipped to make them.

Progress in Soil Mechanics has Lagged.

The rapid adoption of the indicator and the compaction tests suggest that the profession is only too eager to include as working tools those soil examination procedures which have practical value in the fields of design and construction. Its failure to include the consolidation and shear tests is the best proof that the subject of soil mechanics in which data furnished by the consolidation and shear tests are utilized, has not, with few exceptions, as yet emerged far enough from the realm of research to have general use in practice.

This is indicated by opinions included in a "Symposium on the Practical Application of Soil Mechanics," *Transactions* A. S. C. E. (14), with special reference to building foundations. Charles Terzaghi (page 1433) states:

"Considering the present state of knowledge in the field of foundations, the settlement observations represent by far the most important type of research. The results of theoretical and of laboratory investigations in this field cannot claim more than an academic value until the importance of errors has been investigated thoroughly and repeatedly."

Reference is made also to discussions by A. Strieff, page 1496 and Jacob Feld, page 1466 (14).

Apparently then, the problem of changing the results furnished by research into workable tools of design now confronts the profession as a whole and furthermore, as an essential step in its solution, observations of the performance of structures in service must be made. By the same token it is equally important that yardsticks or gauges be set up as a basis of comparison, if the full value of settlement observations is to be obtained.

To illustrate, reference is made to the development of the subject of hydraulics. As a gauge or yardstick, it was accepted that the theoretical flow of water was in accordance with the relation

$$v = \sqrt{2gh}$$

in which

v = velocity of flow
g = acceleration due to gravity
h = pressure head

The actual flow under the many desired conditions met in service was then determined by observation and experiment, after which the basic relation was modified to read

$$v = C\sqrt{2gh}$$

in which the coefficient C expressed the ratio of the actual velocity to that indicated by the theory, and was found to have values of 0.5, 0.7 and numerous others less than 1.

Possibly progress in soil mechanics especially as applied to embankment construction can be expedited by following much the same course, utilizing as yardsticks estimates of performance computed by means of usable test data substituted in expressions of current theories of stress distribution in loaded soil. Such procedure utilized by the various State highway departments, supplemented by observations of the performance of structures after construction would make every highway bridge, retaining wall and embankment a potential source of the desired information. This combined with corollary large scale investigations of earth masses under controlled conditions of accelerated test should quickly furnish the needed values of the correction coefficient C.

Principally to assist in such possible approach to the design and construction of embankments, a review of suggested theories is presented. In its perusal, cognizance should be taken of the fact that soil mechanics is not an exact science. The theories of stress distribution have for their basis the assumptions that soil en masse is homogeneous, isotropic and

either entirely elastic or entirely plastic. As a matter of fact, it is neither homogeneous nor isotropic, and its deformations depend upon some complex combination of the properties of elasticity and plasticity, and not on either of them independently. Therefore, the extensive use of approximations and short cuts to reduce the effort involved in the applications of the theories seems justified. The review considers two types of soil deformation: those due to consolidation of the soil, and those which depend upon its lateral displacement.

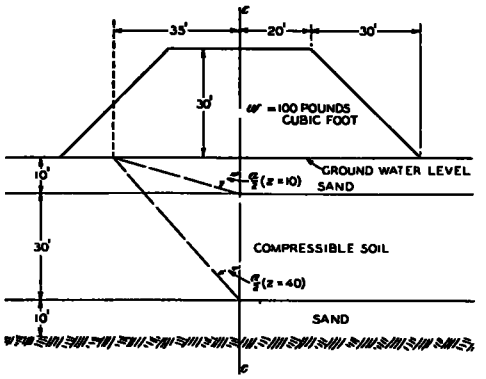


Figure 1. Soil Profile and Embankment Cross Section

SETTLEMENT OF EMBANKMENT DUE TO CONSOLIDATION OF THE UNDERSOIL

This material supplements the report "The Trend of Soil Testing" presented at the Eighteenth Annual Meeting of the Highway Research Board with special reference to the discussion, "The Consolidation Test."

The Problem.

Let it be assumed that an embankment of the cross section shown is to be constructed on the soil profile, Figure 1. Let it be assumed further that the natural soil is sufficiently old to have attained complete consolidation due to weight of the soil; that the weight of the immersed undersoil equals 60 lb. per cu. ft.; and that the fill material will weigh 100 lb. per cu. ft. It is considered that the sand is incompressible and therefore that only the compressible layer of soil will undergo consolidation when the fill is placed.

Computation of the Total Settlement.

The first step is to compute the inactive pressures or those under which the soil has been compressed and which consequently are no longer effective for producing further consolidation. In the case of the profile, Figure 1, the inactive pressure at the top of the compressible layer becomes

$$10 \times 60 = 600 \text{ lb. per sq. ft.}$$

TABLE 1

CONSOLIDATION DATA

Vertical pressures, lb. per sq. ft.	Voids ratios e	Average voids ratios
Inactive at top boundary..... 600	1.47	$1.33 = e_i$
Inactive at bottom boundary.....2400	1.19	
Total at top boundary.....3573	1.09	$1.05 = e_f$
Total at bottom boundary.....4716	1.02	

and at the bottom of the compressible layer

$$40 \times 60 = 2400 \text{ lb. per sq. ft.}$$

These are recorded in column 1 of Table 1 as shown.

As a second step, the consolidating pressure or that which will be produced by the embankment when constructed, is determined by means of the Boussinesq formula (13) for pressures, p_z, along the center axis beneath a uniform strip load. It is as follows:

$$p_z = \frac{p}{\pi} (a + \sin a)$$

in which

p = pressure in excess of the inactive pressure exerted by the embankment at its base.

a = twice the angle in radians between the vertical and a line from the side edge of the uniform strip to any point along the center line at a distance z below the base of the strip load.

By substitution

$$p_{(z=10)} = \frac{3000}{3.142}(2.584 + 0.528)$$

$$= 2973 \text{ lb. per sq. ft.}$$

In the same manner, the pressure at the intersection of c-c and the bottom of the compressible layer is found to be 2316 lb. per sq. ft.

The consolidating pressures added to the inactive pressures give total pressures

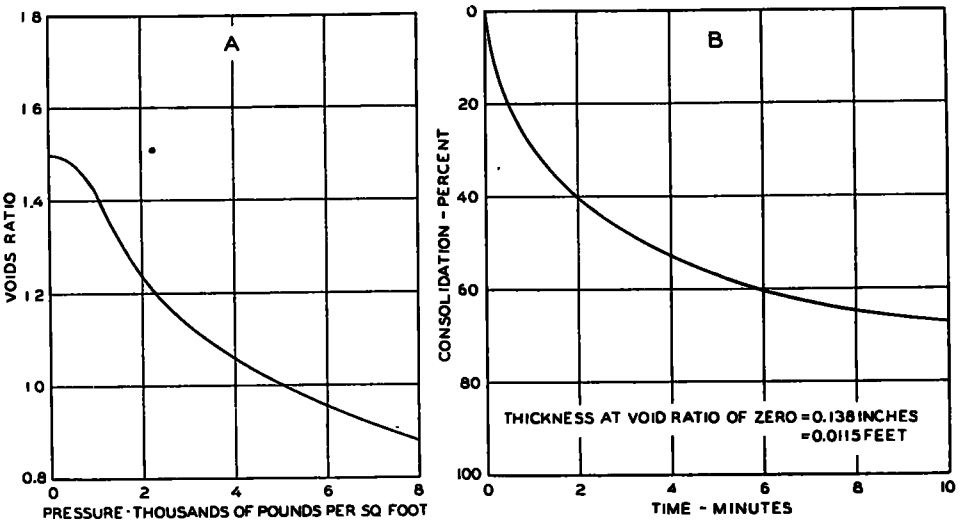


Figure 2. Consolidation Characteristics of Undisturbed Sample

In the case of the embankment, Figure 1, the side edge of the strip load is taken as the edge of a rectangle equivalent in height and area to the embankment cross section (15).

With p equal to 3000 lb. per sq. ft., the consolidating pressure exerted along the center axis (c-c) at the top of the compressible layer is computed as follows:

$$\text{Tangent}\frac{a}{2} = \frac{35}{10} = 3.5$$

Arc tangent $3.5 = 74.06°$

then

$$a = 2 \times 74.06° = 148.1°$$

$$= \frac{148.1 \times \pi \text{ radians}}{180} = 2.584 \text{ radians}$$

$$\sin a = \sin 148.1° = \sin 31.9° = 0.528.$$

of 3573 lb. and 4716 lb. per sq. ft. acting at the top and the bottom boundaries respectively of the compressible layer, and these also are recorded in Table 1.

Reference is now made to the data, Figure 2, furnished by consolidation tests performed on an undisturbed sample of the compressible soil. From the pressure-voids ratio curve, Figure 2-A, the voids ratios at the pressures listed in column 1, Table 1, are determined and placed in column 2 as shown. The voids ratios of the inactive and the total pressures are then averaged separately as e_i, the initial voids ratio, and e_f, the final voids ratio, and placed in column 3.

With these data the total settlement Q

may now be determined from the expression

$$Q=\frac{e_i-e_f}{1+e_i}\times D$$

in which

D = thickness of compressible layer at e_i = 30 ft. in this case

By substitution

$$Q=\frac{1.33-1.05}{1+1.33}\times 30=3.6 \text{ ft.}$$

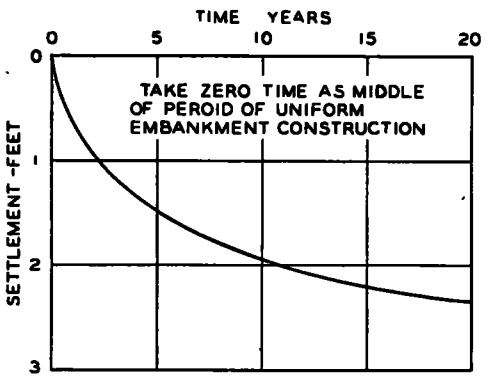


Figure 3. Estimated Rate of Settlement of Embankment

Rate of Settlement.

From the time-consolidation curve, Figure 2-B, the percentages of consolidation which the soil sample underwent at several arbitrarily selected times are listed in columns 1 and 2, Table 2. These percentages of the total consolidation of 3.6 ft. of the compressible layer may then be determined as in column 3. The estimated periods of time for the compressible layer to consolidate the different amounts shown are then computed from the relation

$$t_D=t_d\left(\frac{D_0}{d_0}\right)^2$$

in which

t_D = time required for the soil layer to consolidate a given percentage.

t_d = time required for the soil sample to consolidate the same percentage.

d_0 = thickness of soil sample at voids ratio, e = 0. It is furnished as part of the test data. See Figure 2-B.

D_0 = thickness of soil layer at e = 0. It equals $\frac{D}{1+e_i}$, equals $\frac{30}{1+1.33}$ = 12.9 ft.

Thus the time for the soil layer to consolidate the same percentage that required 1 min. for the soil sample is

$$t_D=1\times\left(\frac{12.9}{0.0115}\right)^2=1{,}260{,}000 \text{ min.}$$
$$=875 \text{ days}=2.4 \text{ years.}$$

Then the time corresponding to 2 min. is 2×2.4 = 4.8 years, etc. This gives column 4 Table 2 which with column 3 is used to construct a time-settlement graph for the embankment as shown in Figure 3.

TABLE 2

RATE OF SETTLEMENT DATA

Soil sample data Figure 2-A		Soil layer data	
Period of consolidation	Percentage of consolidation	Amount of consolidation	Period of time required
Min.	%	Ft.	Years
¼	12	0.43	0.6
½	20	0.72	1.2
1	29	1.04	2.4
2	40	1.44	4.8
4	53	1.91	9.6
8	65	2.34	19.2

The validity of such an analysis is indicated by work reported in *Public Roads,* February 1936 (16). Here the moisture contents of samples obtained from the soft foundation soil beneath a hydraulic fill placed at Four Mile Run on the Mount Vernon Memorial Highway were determined in the laboratory and were also computed according to the theory of consolidation. The results given in Table 3 show that the average of the computed values varied not more than

5 percent from the average of the determined values.

The foregoing gives the complete settlement analysis step by step for compressible undersoil with two drainage faces as in Figure 1. If the compressible layer has but one drainage face instead, the periods of time required for given percentages of settlement computed on the basis of two drainage faces must be increased by multiplication with the coefficients given in Table 4.

THE INFLUENCE OF SHEAR RESISTANCE ON THE STABILITY OF SOIL

The following discussion supplements material included under the title "The Trend of Soil Testing" presented at the Eighteenth Annual Meeting of the Highway Research Board, with special reference to the discussions of "The Theory of Shear Tests" and "Only Part of Soil Strength is Usable." Reference is made also to information on direct shear tests presented at the annual meeting of the American Society for Testing Materials, 1938 (17), and on the stabilometer tests published in the September 1939 issue of *Public Roads* (18).

The data, Table 5, are presented principally to illustrate the influence of the cohesion, c, and the angle of internal friction, ϕ, of soil on its supporting value, etc., computed from theory based upon Coulomb's classical conception of its shear resistance.

The deformations, m, and the corre-

TABLE 3

MOISTURE CONTENT DATA—FOUR MILE RUN INVESTIGATION

Undisturbed core number	Moisture content of the foundation soil	
	Determined by test	Computed from theory
	Percent	*Percent*
1	108	123
2	76	60
5	125	123
6	79	77
9	105	86
10	116	109
Average	101	96

TABLE 4

EFFECT OF BOUNDARY CONDITIONS ON RATE OF CONSOLIDATION

Consolidating pressure at drainage face ÷ consolidating pressure at impervious face	Time of consolidation with one drainage face÷time with two drainage faces for the following percentages of consolidation:					
	5	20	40	60	80	95
0	50	13	7.0	5.4	4.5	4 3
0 2	17	9.2	6.4	4.8	4 4	4 2
0.4	9.2	6.8	5 6	4.5	4 3	4 2
0.6	6.0	5.4	4 8	4.4	4.2	4 1
0.8	4 6	4 5	4 4	4.2	4.1	4.1
0.9	4.2	4.2	4.1	4 1	4.0	4.0
1.0	4.0	4.0	4.0	4.0	4.0	4 0
1.5	3.5	3.0	3.6	3.7	3.8	3 9
2	3.1	2 5	3.3	3.5	3 7	3 9
3	2.6	2.1	2.9	3 3	3 6	3.8
5	2.1	1.7	2.5	3.0	3.4	3 8
10	1 7	1.4	2.1	2.8	3.3	3 7
∞	1.6	1 1	1.6	2.3	3 1	3.6

shown as 3 in Table 5 and in which

L = total lateral pressure per foot width of wall.
h = height of wall.
w = weight of earth per cubic foot.

The passive earth pressure P is expressed by the formula

$$P = h\left(\frac{wh}{2}\tan^2 a + 2c\tan a\right)$$

shown as 4 in Table 5.

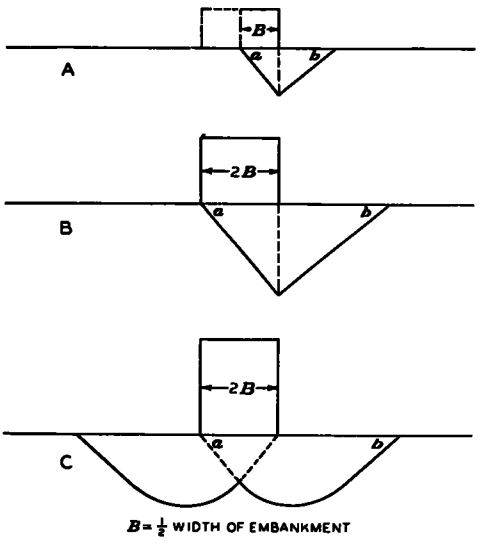


Figure 6. Surfaces of Slip Under Embankments ($\phi = 10$)

Supporting Value of Soil.

Formulas 3 and 4 it will be noted consist of two parts, a first term, which depends upon the weight of the earth in the wedge and is independent of the cohesion, c, and a second term which depends upon the cohesion c and is independent of the weight of the earth in the wedge.

If now we refer to the computed values of both L and P in Table 5, it will be noted that those for P are much larger than those for L. The additional pressure, q, which can be applied to the surface of the earth producing active pressure, to just equalize the excess of P over L is obtained by means of the formula

$$q = (P - L)\tan^2 a.$$

By substitution of the values of P and L, and simplifying, there results the expression

$$q = wB\frac{\tan^4 a - 1}{2\cot a} + c\frac{2\tan a}{\cos^2 a}$$

which is shown as formula 5, Table 5. (See diagram, Fig. 6-A) and in which $B = \frac{1}{2}$ width of loaded strip.

If it were desired to support a load, q, which would cause the active pressure to exceed the passive resistance, it would become necessary to apply a surcharge, q_1, to the surface of the earth furnishing the passive resistance, the magnitude of q_1, being given by the expression

$$q_1 \tan^2 a = q\tan^2 b + L - P$$

If now this equation is simplified there results the expression

$$q = wB\frac{\tan^4 a - 1}{2\cot a} + c\frac{2\tan a}{\cos^2 a} + q_1\tan^4 a$$

and this will be recognized as an expression of the conception of supporting value which was first published in *Public Roads*, May 1929 (2).

Since the width B, can be taken as one-half the width of a loaded strip, the formula has been suggested for use in estimating the supporting value of the foundation soil of an embankment which fails by breaking in the middle.

If the embankment can be considered rigid enough to settle as a unit but fail by tilting to either side the formula becomes as follows:

$$q = wB\frac{\tan^4 a - 1}{\cot a} + c\frac{2\tan a}{\cos^2 a} + q_1\tan^4 a.$$

If no surcharge is to be used, only the first two terms of the supporting value formulas are used. If the soil is cohesionless and without surcharge, only the first term is used; and if the soil is frictionless and without surcharge, only the second term.

Formula 6, Table 5, gives values of supporting value for an embankment which fails as a unit by tilting, and where no surcharge is to be used. See diagram, Figure 6-B.

When it can be considered that the embankment will act as a unit but settle vertically without tilting, there would be warrant for using twice the values obtained by means of formula 6, but in such case the more conservative values furnished by use of the Prandtl formula (15, 20), No. 7, Table 5, seem more appropriate. See diagram, Figure 6-C.

SETTLEMENTS OF EMBANKMENT DUE TO LATERAL DISPLACEMENT OF THE UNDERSOIL

If the distortion of soil required to build up values of c and ϕ behind retaining walls and beneath embankments were proportional to the deformations required in the test samples (1, 17), expressions for use in estimating the earth movements indicated in Figure 5, such as the following could be derived:

For wall moving outward due to active pressure—

Average horizontal displacement of wall in inches (Formula 8, Table 5)

$$L_l = .06mh \sin^2 b$$

Average settlement of earth behind wall (Formula 9, Table 5)

$$L_v = .06mh \sin b \cos b.$$

How much weight can be given the values of the deformations computed from these formulas as shown in Table 5 is not known. The principal purpose of including them here is to give at least a qualitative conception of the slow distortions which stressed plastic soil undergoes in the building up or mobilizing of its shear strength, and thus explain the slow sway or displacement of retaining walls and bridge abutments which have been designated as "creep." Also, they serve to emphasize the risk involved in the use of shear resistances in design which require large deformations of the soil to mobilize.

Regardless of the values shown in the last column, Table 5, which are based on the ultimate c and ϕ of the soil, structures built on the soil can safely undergo only a limited amount of settlement, L_v, and retaining walls, but a limited amount of lateral displacement L_l. If it were considered that the allowable values for L_v and L_l were those produced by sample deformations of m=2 percent, then the c and ϕ with the corresponding computed values, Table 5, would be used. Such c and ϕ for the soil used in the foregoing demonstrations equal approximately $\frac{1}{3}$ the ultimate values and, therefore, it can be considered that in their use a factor of safety of 3 with respect to the total strength was employed.

Design of Slopes.

The safe slope of embankments placed on soft undersoil depends upon the shear resistance of the foundation soil; and of those placed upon good undersoil, upon the shear resistance of the fill material.

An expression which shows the relation of safe slope of embankment to the shear resistance of relatively thin layers of soft undersoil was published by the Ohio Department of Highways, 1937 (25). With modifications it is as follows:

$$S = \frac{Dw_s}{2c + Dw_u \tan \phi}$$

which is shown as formula 10, Table 5, and in which

S = slope = ratio of run to rise.
D = thickness of soft undersoil.
w_s = unit weight of fill material.
w_u = unit weight of undersoil.

In computing the values of S for the three thicknesses of the soft undersoil, Table 5, it was assumed that $w_s = w_u = 100$ lb. per cu. ft.

For determining the safe slopes with respect to the shear strength of the em-

bankment material, a very laborious graphical method based upon the conceptions of Fellenius and others was described in *Public Roads*, December 1929 (19). Later the method was modified and very much simplified by D. W. Taylor (26). A further simplification by the junior author is made possible by use of the graphs, Figure 7.

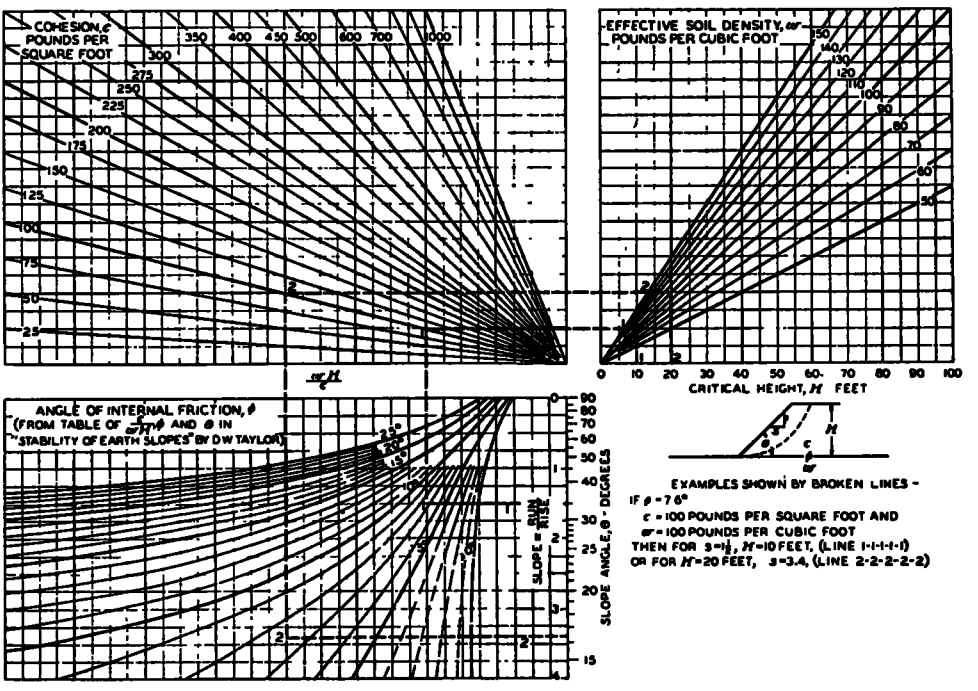


Figure 7. Chart for Critical Height of Slopes

When w, c and ϕ are known, the critical height of an embankment for any desired slope may be determined by working around through the three graphs clockwise; and the critical slope for any desired height, by working through the graphs counterclockwise. Values obtained in this manner for critical heights at a slope of $1\frac{1}{2}$:1; and the critical slopes for a height of embankment, H=20 ft., are shown in Table 5.

Additive to the information furnished on the effect of c and ϕ relative to the safe slopes of embankments, the graphs, Figure 7, were introduced here to illustrate the enormous saving in time spent on computations, made possible by the use of such methods, which are equally applicable to the use of all the relations shown in Table 5, and to the others commonly used in soil work.

SUMMARIZATION

Thus it can be said that enviable progress has been made in the utilization of the indicator and the compaction test data. And the outlook in the field of soil mechanics as applicable to the construction of embankments is not so dark. With the exception of simplifications in the apparatus, the essential features of the consolidation test are for all practical purposes the same as when originally proposed by Doctor Terzaghi, and may

well be standardized now for highway purposes. Within the range of the uniformity of the soil, it is indicated that for those conditions of soil profile and loaded area which approach the conditions under which the samples are tested in the laboratory, estimates of settlement of structures due to consolidation of the undersoil made in accordance with the theory of consolidation, are entirely valid, and are especially applicable to embankments due to their broad loaded areas. As the ratio of loaded area to depth of compressible soil decreases, the breach between field and test conditions widens, and the validity of the settlement estimates can be expected to correspondingly diminish.

Progress in shear tests has consisted principally in the development of suitable apparatus and a better understanding of the errors involved in the improper interpretation of test data. It is evident that the time is not yet ripe for the standardization of shear test procedures, although ideas are rapidly crystallizing with respect to simplified types of direct shear and stabilometer test apparatus. The validity of formulas based on Coloumb's conception of shear resistance cannot be determined until more adequate information is obtained on the performance of stressed earth masses.

It is appreciated that the use of partial values of c and ϕ introduces an error, for, in the theories expressed by the formulas a condition of ultimate failure of the soil is assumed. On the other hand, values of c and ϕ indicative of the ultimate strength of soil obviously could not be used because of the prohibitive deformations of the soil involved, even if they could be obtained. But no criterion for the determination of the c's and ϕ's indicative of the ultimate failure of samples during test has as yet been established, whereas, there is no difficulty in obtaining c's and ϕ's indicative of shear resistances of the samples at equal deformations.

It is expected that charts such as shown in Figure 7 to substitute for the formulas in the making of computations will greatly expedite a more general use of the theories. The next step is to encourage more controlled investigations such as have been in progress at the University of Michigan (27) and the Iowa State College (28, 29); and the applications of the theories in practice as was so well exemplified by the Bay Bridge approaches and the San Luis Obispo projects in California (1). Along with these the study of building settlement by Charles Terzaghi (14) and Gregory P. Tschebotareff (30); and of large bridges by W. P. Kimball (31) are furnishing valuable supplementary information.

In conclusion a word should be said on the desirability of utilizing controlled methods of compaction in embankment construction. Investigations disclose that embankments constructed under rigid specifications but before the advent of moisture control have densities ranging from 70 percent up to 100 percent of the Proctor maximum. By means of shear test data obtained in the Administration's laboratory and the chart Figure 7, it can be shown that unusually bad soils which have critical heights of not more than 10 ft. when compacted at 77 percent Proctor maximum may be used in embankments up to 60 ft. high at 90 percent, and 120 ft. high at 100 percent Proctor density. It is further indicated that the attainment of the required high densities in embankment construction does not necessarily mean increase in the cost of their construction.

The advantages of this method of construction are summarized by Thos. E. Stanton (32) as follows:

"Standard California practice requires that embankments be constructed and consolidated in layers not more than 8 in. thick before compaction and that each layer be consolidated by rolling to a relative compaction of not less than 90 per cent (California maximum) before subsequent layers are placed. By this method com-

bined with rigid control over the moisture content it has been found possible to construct embankments to any height without subsequent appreciable settlement.

"Except for the fact that the specifications provide that no adobe shall be placed in embankments above one foot below the profile grade, there is no restriction in California specifications regarding the nature of the material which can be used.

"It has been found possible to secure relative compactions of 90 to 100 per cent with even the difficultly consolidated California adobe soils of the A-7 type, provided the construction procedure laid down in the specifications is rigidly followed.

"A number of major projects, each with many large fill and embankment quantities totalling in excess of a million cubic yards have been constructed from materials consisting largely of clay-shale of the A-7 soil type. Relative density in excess of 90 percent and a dry density of approximately 125 pounds per cubic foot were secured. One such fill, 130 feet high on the Cuesta Grade was constructed in 1937. Notwithstanding a severe wet winter subsequent to the construction of the fill, no appreciable settlement has taken place during the first year.

"Three projects constructed in 1929 in accordance with the California specifications have been under observation for nine years. The embankments ranged from 30 to 110 feet in height. During nine years of service no appreciable settlement of these large fills has occurred."

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DISCUSSION ON SOILS INVESTIGATIONS

MR. J. J. FORRER, *Virginia Department of Highways:* How many States are putting into practice the method described by Mr. Hogentogler?

MR. HOGENTOGLER: According to the compendium last year there are 24 States who reported making the Proctor tests. Both California and Ohio construct fills by controlled compaction methods.

How many of the other States that reported facilities for making compaction tests actually carry out all the details in construction, I do not know, but this I believe is true—that even though they have not gone so far as the two States mentioned above in actual control, just an appreciation of the idea that is back of these tests has done wonders toward improving the stability of fills in ordinary construction.

MR. FORRER: What are the additional costs on a yardage basis due to using the optimum moisture in fill construction?

MR. HOGENTOGLER: I cannot answer that but a discussion came up a year or two ago, the trend of which was that the contractor saves money by utilizing controlled methods of compaction during construction. Among other things, the grades constructed with sheepsfoot rollers have been compacted so highly that he does not have to wait for them to dry out after rain. Professor Casagrande brought out some illustrations in New Hampshire in which it cost the contractor less to utilize the compaction method than if he had not used it.

MR. J. J. PREECE, *National Park Service:* Maybe I can add a little more information to the last question. We are now constructing or reconstructing the C & O canal and in that we have some embankment work to do. The specifications were written up with laboratory control for the embankment construction. The first reaction of the contractors who were expecting to bid on that job was complete consternation. They threw up their hands, one of them who had done a great deal of work refused to bid at all and they came back at me and asked me if I had not written up something that

was just so impractical that we could not expect to carry it through. I did not think that we had. As a matter of fact I think we had written up an extremely liberal specification.

We required no particular water control. We provide the Water Content—Compacted Unit Weight relation curve, and we tell the contractor we want compaction to a certain minimum unit weight. Now it is well known that for a given water content you get a higher compacted unit weight by more work. The unit weight which is obtained with a certain amount of work at optimum water represents the least work that must be applied in order to obtain that unit weight. We specified only the minimum compacted unit weight and told the contactor—you can change your water content to get this with less rolling or you can leave your water content and get the compaction by increased rolling, whichever is cheaper in any particular case. That I believe is rather a liberal provision. Finally the bids came in. To get the bids it was necessary to do some missionary work and convince the contractors that we were not going to tie them up in a knot. They finally bid on the work, relying on what we had told them we would do. The work has been going on now for some time and recently I asked the contractor how he felt about this control and he is quite enthusiastic about it.

I don't believe that any contractor who has any conception of what this soil mechanics laboratory control is all about, or has any idea of what he is shooting at and what the laboratories are trying to do, will feel that he is being put to any disadvantage. As a matter of fact, we are building all over the United States—although not by contract—most of it is CCC work. While we have not run into the bidding factor, we have encountered the opposition of men who have been constructing for a long time. They feel that they know how to do it and do not want any waste for refinements placed in their path. They do not want any fancy theories. Nevertheless, I don't know of a single one who has tried it who is not completely enthusiastic about it and believes that it not only permits him to keep a better control, and do better work, but also results in definitely lower costs.

C. M. UPHAM, *American Road Builders Association:* Moisture control was not specified on the Pennsylvania Turnpike. It was later recommended that moisture control should be used and when it was found from the contractor how much this additional operation would cost, it was found to be prohibitive. The ridiculous prices which were asked by the contractor made it impossible to use the optimum moisture content method. The contractor wanted from two to eight cents per gallon. You could buy gasoline cheaper than that. The high price was due to the fact that the moisture requirement was not included in the specifications in the beginning.

PROF. M. G. SPANGLER, *Iowa State College:* I have in mind a matter which might have a place in this discussion; that is the problem of subsidence of foundation material under high embankments. No matter how well the embankment has been compacted, we are still up against the matter of compression or consolidation of the natural ground or foundation material. In some cases that is a real factor. I myself have measured subsidences under an embankment 65 feet high which amounted to 2 feet, 8 inches at the maximum point.

I recall a story which Dean Marston tells in connection with his work on the Board of Review of the Mississippi Flood Control Works, in which he cited an example of a levee under which the subsidence had been as much as 80 feet. This of course is an extreme case, but

subsidences measurable in feet are not uncommon and as embankments become higher due to reduced costs of earth-moving and grade and alignment requirements for adequate sight distances, the problem of subsidence is bound to become more noticeable.

MR. HOGENTOGLER: In that connection see the 1937 Civil Engineers Symposium of practical application of soil mechanics. The point is made by one contributor that at the present time about 85 percent of all failures of embankments are due to foundation trouble.

MR. V. J. BROWN, *Roads and Streets:* As I understand soil mechanics in its application, we have to analyze each type of soil independently. When we come to build a big embankment and we dig down into the cut out of which it is to be built, we are going to find several types of soil, shale and rock. This is especially true for deep cuts—and these are the kind that furnish the material for big embankments. It occurs to me that at the present time we are building our embankments upside down. We are taking the weathered soil from the top of the cut and putting it on the bottom of the fill. When we strike hard rock near the bottom of the cut, we put that on the top of the fill. It we were to shoot the full face of the deep cut, we would get a mixture of materials to place in the fill that would be uniform. What I am wondering about is, could our soils mechanics analyses be applied to that method of construction giving us definite results? In other words, could soil mechanics embankment control methods be applied when a construction method is used in which the full face of cut is shot and the mixed materials hauled into the embankment?

MR. HOGENTOGLER: Assuming that you have a combination of soils in which the optimum is changing and you place your fill at the optimum moisture content of one of the soils, there is the likelihood that the others will be adequately compacted.

QUESTION: In line with Mr. Brown's query, approximately what size soil do you think was used?

MR. HOGENTOGLER: The Proctor test has to do with clay soils and is applicable whenever there is sufficient clay to float the granular material.

MR. V. J. BROWN: Has there been a correlation established in compaction between the tests on the soil as you make it and the results in the field where mixed soils, such as topsoil and rock, or shale and topsoil, etc., constitute the embankment?

MR. HOGENTOGLER: The compaction tests are used in the construction of topsoil and stabilized roads, although there is no relation between the test results and the number of trips of the compacting equipment.

MR. V. J. BROWN: I have talked with manufacturers in regard to compaction. Whenever I talk about the inefficiency of the sheepsfoot roller they come back with the old argument that for every soil there is a definite optimum water content which is related to the weights of the rollers and the amount of work done. There is also a different compaction density for different equipment and I maintain that we should eliminate the sheepsfoot roller in favor of some more efficient unit. We should have a unit that would give results comparable to our standard test method of compaction, which would be some kind of a machine that would hit every square inch of the surface uniformly.

MR. PREECE: The first thing that any laboratory is going to do with regard to

soil mechanics is to set up for itself a procedure which is correlated to the work it is going to direct; that is the first requirement. The laboratory that is off by itself and is turning out page after page of data and is not interested in the job, is not going to get anywhere. We started some time ago to determine a method by which we could be sure our laboratory work was definitely correlated with our work in the field. I can tell you this and we have been using it for several years. In the meantime we have probably built about 45 to 50 dams, some of them going as high as 125 feet. We do have a definite relation, that is, on the curve that we give to the field.

PROF. DONALD W. TAYLOR, *Massachusetts Institute of Technology:* During the past decade there has been much progress toward a better understanding of the factors upon which the shearing strength and related characteristics of clays depend. Also there has been steady improvement in the types of apparatus which have been devised for determining the properties of clay soils. However, many of the complex phenomena which affect these properties are still but partly understood. This is especially true of the shearing strength, the most complex of soil properties and the one to which this discussion will be mainly devoted.

During the past few years numerous soil laboratories have been established by agencies anxious to make practical use of the latest developments in soil mechanics. Demands for the setting up of standardized procedures for various soil tests have been growing. At the same time there has been much agitation for the establishment of rules for interpreting shear test data and for recommended safety factors to be used in design.

There are two strongly contrasting beliefs with regard to the advisability of standardizations and the setting up of such rules. Regardless of which belief an engineer subscribes to, he will do well to understand both points of view and to keep both in mind in discussions of the subject.

First is the belief that in such an involved subject as shear in clays there are so many variables upon which the results depend, that, so long as new ideas are continually being developed which lead to alterations of those they replace, standardization of testing methods is undesirable. Any simple testing method cannot truly reflect all possible conditions of the many variables. Also, standardization may, in many cases, place a false appearance of dependability on test results. It will probably be generally agreed that for the best ultimate understanding of a complex problem, there is much logic in this viewpoint.

The second belief is frequently spoken of as a practical viewpoint. Engineering practice cannot wait for the development of perfect methods but demands at any time the best procedure which can at the time be set up. Trial in use will point out certain flaws quicker than can be found in any other way.

It is probable that most engineers will agree that both points of view are somewhat extreme and that the great majority of cases must be considered separately, each on its own merits. It is also probable that the standardization of many of the common soil tests is on the whole desirable. However, the time has not yet been reached when the standardization of shear tests on clays can be expected to be at all permanent and it is believed that the disadvantages of attempts toward standardization greatly outweigh the advantages. This is well illustrated by the current situation with regard to shearing strength investigations by triaxial or cylindrical compression apparatus, a type which has jumped into prominence in the last two or three years. Rapid strides have been made in methods of investigation on sands as shown

by the new concepts of critical void ratios. Extension of this work to clays has only just begun.

One of the questions most frequently asked in soil mechanics work is the following. When tests for the determination of shearing strength have been conducted on samples from a given site, how does one determine from the test data the shearing strength values which should be used in the design or analysis of an embankment at the site, and if a design what factor of safety should be used? Surely, if a definite and dependable list of rules could be given they would be of great help to many engineers. Unfortunately, the question is much too complex to permit the setting up of definite general rules. If general rules were possible they could hardly be covered in one short paper. As has often been pointed out in foundation work there are conditions which are peculiar to any given job and as a general policy, each job unless it be a relatively simple one must be treated individually.

Since it is not possible to give a treatment which is as definite and general as would be desirable, the following discussion is limited to a few remarks on the subjects of factors of safety and factors affecting the shearing strength of clays.

Since failures may occur in many different ways, there are many possible types of factor of safety. In stability analyses of embankments it is especially important that the term "factor of safety" should not be used until its meaning for the given case is clearly defined. The question arises as to what shall be the criterion of failure; shall failure be associated with the condition wherein the most highly stressed point just reaches its limit of strength, or shall it be considered that failure has not occurred until the point which first attains the failure state has undergone further strain, at constant or decreased strength, until all reserve of strength of surrounding points has been expended?

Both concepts have been widely used. If a soil is of sufficient strength so that its most dangerously stressed point has a shearing strength which exceeds its shear stress by 50 percent, it may be said to have a "point factor of safety with respect to shearing strength" of 1.5. Complete failure in shear, however, would not occur by merely overstressing one point. Rather, all strength along a continuous surface would have to be overcome and the factor of safety which is in commonest use in stability analysis is sometimes called the "factor of safety with respect to total strength." It may be described as the ratio between total shearing strength and total applied shear on the continuous surface most susceptible to failure.

Field investigations by the Swedish Geotechnical Commission and other organizations on actual slides, supplemented by laboratory tests for shearing strength, have pointed out that it is seldom possible to use large factors of safety with respect to total strength in embankment design. Typical practical values are of the order of perhaps 1.1 to 1.5 while if higher values are attempted, prohibitively large increases of yardage may result. An illustration given by the writer at the Highway Research Board meeting of a year ago indicated this type of situation for a foundation failure of a highway fill. In this case a factor of more than 1.2 in the redesigned section would have required excessive yardage.

However, the following illustrates an important point. Let it be assumed that in a typical case the factor of safety with respect to strength is $1\frac{1}{3}$ and that in this same case the shearing strength determinations have a probable error of about 30 percent. This is, perhaps, a rough idea of the usual accuracy for tests on some types of soils. Here it is seen that the margin of safety is of the same order of magnitude as the probable error in the strength determinations, a more or

less typical situation. This indicates the very practical fact that efforts along lines such as combined shearing tests and field investigations of failures, aiming at improved understanding of shearing strength values, are much more productive than are efforts to decide what factor of safety should be used.

The factors which affect the shearing strength of clay samples are numerous and complicated but they may be grouped under the following headings:

1. Porosity
2. Pressure
3. Strength due to natural structure
4. Condition of consolidation and other time factors
5. Degree of progressive action.

A thorough discussion of these items would fill a large textbook. The following is an attempt to outline the entire complex subject in a single paragraph.

While porosity has long been known to be an important factor, research by M. J. Hvorslev has demonstrated that an expression in terms of the porosity is also capable of expressing the effects of the past stress history of the sample. The total pressure has always been recognized as a factor of importance. Also the strength which is given by the natural structure and which may be largely lost if the structure is disturbed in sampling is a most important item. As a sample undergoes shear, the shear distortion in itself causes disturbance of structure and to a degree softens the sample, thus transferring part of the pressure on the sample into excess pressure in the pore water. Therefore, in addition to the first three factors mentioned, there must be added the fourth, "degree of consolidation." This factor will in turn depend upon many things, including properties of the natural structure, the speed at which the test is run and the size of the sample and other factors influencing the facilities for drainage of pore water from the sample. Other time effects such as a resistance akin to viscosity may occur, especially if shear is forced to take place quickly. Lastly, unless the soil is under uniform stress and strength conditions throughout, which probably is never the case in nature and which holds only approximately even in the best of laboratory testing methods, there will be progressive action, or weakening of strength caused by load concentrations which successively cause overstressing which passes from point to point.

The claim has sometimes been forwarded that soil mechanics can remove guesswork from foundation and embankment problems. To a degree this is true but there must always be a much larger degree of uncertainty in problems involving soils than there is in most engineering problems. In removing guesswork there are two ways in which soil mechanics has value. First, by a clearer understanding of the physical phenomena which govern the action of soils, designing becomes more rational even when soil testing is not resorted to for each specific problem. Soil testing in the form of fundamental research, however, contributes greatly to this first item which could be called qualitative soil mechanics. Secondly, laboratory determination of soil properties for use in analysis and design has proven to be of practical value in some types of problems. These two factors apply not alone to soil mechanics; a similar situation exists in the closely related field of geology. Development of both factors is in every way desirable. It is idle to speculate as to which is more important as both must be carried on together for best results toward practical and sound development.

STRESSES UNDER CIRCULAR LOADED AREAS

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SYNOPSIS

A complete solution of the problem of stresses under a uniformly loaded circular area at the surface of a semi-infinite elastically isotropic material was published by A. E. Love in 1929 and by S. D. Carothers in 1924. The general conclusions and numerical computations of the two investigators are in good agreement. The surface of partial maxima principal stress differences was found by Love to extend down to a depth of about 0.7 of the radius of the loaded circle. The greatest principal stress difference is under the perimeter at the surface. Certain approximate methods of computing the vertical and shearing stresses are described. The results are suggested as a rough guide in experimentation involved in the study of the design of flexible highway surfaces.

The main purpose of this paper is to bring to the attention of engineers the fact, that a precise analysis of the complete system of stresses under a uniformly loaded circular area at the surface of a semi-infinite, elastically isotropic material has been in published form for more than ten years. Another purpose of the paper is to indicate approximate methods of checking numerical values of the stresses that have been computed by the precise methods. Still another purpose is to indicate the possibility of using the results of the analyses as a rough guide in experimentation involved in the study of the design of flexible types of highway surfaces. In this latter connection, it is realized generally that the condition of elastic isotropy does not exist in the mass of material under an automobile tire. Hence, it would be a mistake to attempt to apply directly the analytical results and the fact that they can serve only as a rough guide in experimentation needs to be emphasized.

Engineers are confronted with many earth problems for which no theory other than the one of elasticity has been proposed and the treatment of earth problems apart from adequate theory is likely to lead to as many solutions as there are varieties of earth. The experimental study of such problems without reference to any theoretical basis whatsoever is an aimless procedure at the best.

The problem of computing stresses at any point within a semi-infinite, elastically isotropic mass produced by a uniform load over a circular area at the plane boundary has been completely solved by A. E. H. Love (1)[1] and S. D. Carothers (2). The results of these two investigators are in general in good agreement and suggest a means of estimating stresses under wheel loads, since it has been shown by Teller (3) and Buchanan (3) that the pressure distribution of a pneumatic tire on a flexible type of pavement is nearly uniform.

In such problems as the determination of the rate of settlement (by soil consolidation) of a foundation (see Fig. 1A) the vertical stresses at the upper and lower boundaries of a clay layer are computed from formulas derivable from those of Boussinesq by assuming that the entire mass of earth below the foundation is elastically isotropic. That is, the complications that may be involved, owing to the fact that the underground is comprised of alternating strata of dissimilar materials and is, therefore, not elastically isotropic, are ignored completely. Despite the questionable procedure of assuming isotropy in this case, the analy-

[1] Numbers in parentheses refer to list of references at end.

sis of settlements by consolidation is one of the best established theoretical methods of soil mechanics and the present writer believes that in this instance the assumption of isotropy is as reasonable as any other assumption and is to be preferred because it is in the interest of simplicity.

There no doubt is some refraction of the stress trajectories at the boundary of two dissimilar earth materials, that is,

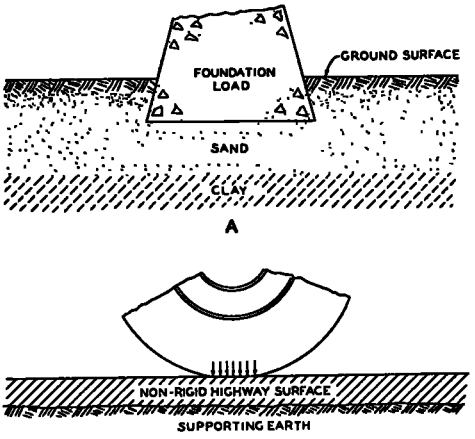


Figure 1

A—Foundation Resting on Sand Underlaid by Clay.

B—Wheel Load Resting on Nonrigid Highway Surface Supported by Earth.

this boundary very likely acts as a plane of discontinuity. But any assumption as to the amount of friction between two layers of dissimilar materials such as clay and sand (Fig. 1A) or as to what extent discontinuity of stresses may exist is likely to be no more valid than the assumption of isotropy in the entire earth mass below the foundation.

For the same reason, the entire mass of materials below a wheel load (Fig. 1B) may be considered as being elastically isotropic.

On the basis of such an assumption, no precise solution can be expected. Nevertheless, the theoretical development meets a definite requirement in that it shows, qualitatively at least, what happens under wheel loads and, therefore, may serve as a guide in planning methods of experimentation. All that is sought is a suggested trace of the transmission of stresses by the wheel load since rigorous results or exact formulas are out of the question.

A. C. Benkelman (4) has presented in considerable detail the present status of our knowledge concerning the design of flexible pavements. It is indeed surprising to note that the conclusions reached by Carothers and Love have not been used in the technical publications discussed in Benkelman's report. However, with practically no exception, the various authors mentioned in his report have based their experimental procedures on theoretical considerations involving numerous simplifying assumptions which are similar in many respects to the classical theories of Love, Carothers and Hencky (5).

The simplest three-dimensional problem considered in the theory of elasticity is the one involving axial symmetry. Hence, the problem of a wheel load on a pavement is simplified by the assumption that the area of contact between a rubber tire and pavement is circular although tests indicate that the contact area is elliptic. Thus an "equivalent circular area" with uniform pressure has been used generally by investigators. The equivalent circle is one having an area equal to the elliptic area of contact and its radius is the square root of the quantity, area of contact divided by π.

The subject of pressure distribution over a circular contact area in connection with foundation design has been discussed at length by Cummings (6), Krynine (7) and various others. The simplest problem is the one of uniform pressure over the contact area. In foundation problems, the actual pressure dis-

tribution depends on the relative rigidities of the structure (or loading member) and the earth mass, and in the light of present knowledge no one knows how to express these relative rigidities in quantitative terms. Another difficulty is that any formula expressing the pressure distribution over the contact area can be only approximately true when the deformations are within the elastic limits of the materials and cannot be applicable in any sense as the deformations increase more and more and become characteristic of those attending plastic yield. If an attempt is made to include all of these departures from simplicity in a theoretical development of the subject, any result would be so hopelessly complicated as to be of very small use to practical engineers.

THE GREATEST SHEAR IS NEAR THE PERIMETER OF A UNIFORMLY LOADED CIRCULAR AREA

The outstanding conclusion of Love and Carothers was that the greatest value of the principal stress difference, S, is very close to the perimeter of the uniformly loaded circular area. This difference is twice the maximum shearing stress at any point, that is,

$$S = 2\,s_{max} \quad \ldots\ldots\ldots\ldots 1$$

Table 1 is taken from Love's article and shows values of $\frac{S}{p}$, p being the unit contact pressure over the circular area, corresponding to various positions of the point Q, Figure 2. The ratio, $\frac{r_1}{r_2}$, of the two radial lines, Figure 2, and the magnitudes of the quantities, angle B, $\frac{z}{a}$ and $\frac{r}{a}$, a being the radius of the circle, z being the depth to the point, and B the angle between r_1 and a, determine the position of any point Q.

As the point Q moves in such a manner that the ratio, $\frac{r_1}{r_2}$, approaches zero as a limit, then according to Love, the value of S depends on the angle B, or in other words, S at the point A depends on the direction of approach of point Q to point A. Love has shown that for B = 71°, the limiting value of S as $\frac{r_1}{r_2}$ approaches zero is 0.723p (Table 1). The $\frac{S}{p}$ values of Table 1 were computed by Love for

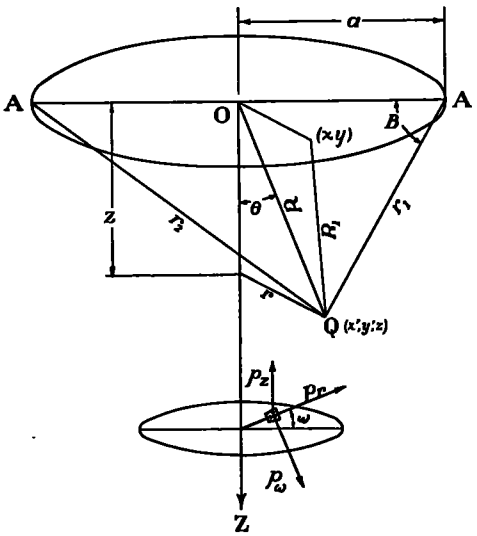


Figure 2. Problem of the Principal Stress Difference, S, under a uniformly Loaded Circular Area.

$\mu = \frac{1}{4}$ and are "partial maxima stress differences," a term that will be considered later in the paper.

From Table 1, the value for s_g, the greatest value of s_{max} at any point Q in the supporting earth is $\frac{0.723p}{2} = 0.36p = \frac{S}{2}$. This is an important fact for the reason that it shows that the greatest shearing stress is at the surface where, due to lack of confinement, there is the least resistance to yield under stress. Love has given in Table 1 more data than are necessary for locating the point Q, Figure 2.

In texts dealing with the theory of elasticity, it is shown that when the point Q (Fig. 2) is restricted to move only along the axis of symmetry, OZ, the greatest value of s_{max} occurs at a considerable depth, z, from the surface. Thus Timoshenko (8) shows that for $\mu=0.3$, the greatest value of s_{max} at any point on OZ is 0.33p and at a depth equal to $\frac{2a}{3}$. The earth at this point is confined in all directions.

Love's solution was obtained by the application of potential theory (9) and involved various elliptic integrals.

Carother's procedure is only very briefly described and therefore requires some discussion. He considers the logarithmic potential (10) of a uniform distribution of matter over a circular area of radius a, expressed by the relation,

$$\psi=\frac{p}{2\pi}\int\int \log(z+R_1)\,dx\,dy \quad \ldots\ldots 2$$

where dx and dy refer to the coordinates of any point x, y, on the surface and R_1 is the distance of this point to the point Q, Figure 2.

By differentiating equation 2 under the integral sign,

$$\frac{\partial 2\psi}{\partial z^2}=\frac{p}{2\pi}\int\int \frac{z}{R_1^3}\,dx\,dy=\frac{pw}{2\pi} \quad \ldots\ldots 3$$

where w is the solid angle (11) subtended at Q by the circular area.

By use of the equations of equilibrium and the compatibility equations (12) all of the stresses at any point, Q, may be expressed in terms of w, the solid angle subtended at point Q. Thus it is found, for example that

$$s_{rz}=z\frac{\partial}{\partial r}\frac{\partial^2\psi}{\partial z^2}=\frac{p}{2\pi}z\frac{\partial w}{\partial r} \quad \ldots\ldots 4$$

$$\text{and } p_z=p/2\pi\left(z\frac{\partial w}{\partial z}-w\right) \quad \ldots\ldots\ldots 5$$

p_z being the pressure at Q that is normal to the horizontal plane and s_{rz} the shearing stress.

Similarly, p_r and p_w (Fig. 2) may be expressed as functions of the solid angle, w.

For a uniformly loaded circular area, the maximum shearing stress is obtained from the expression,

$$s^2_{max}=\frac{(p_r-p_z)^2}{4}+s^2_{rz} \quad \ldots\ldots\ldots 6$$

Love's very comprehensive tables give values for p_r, p_z and s_{rz}, for $\mu=\frac{1}{4}$, at a great many points. Values for $\frac{S}{p}$ in addition to those given in Table 1, may be computed by means of equation 6 and the other tables of Love.

TABLE 1

VALUES OF $\frac{S}{p}$ FOR DIFFERENT POSITIONS OF POINT Q IN FIGURE 2

Values given by A. E. H. Love for $\mu=\frac{1}{4}$[1]

	$\frac{r_1}{r_2}$ tends to zero	$\frac{r_1}{r_2}$=sin 5°	$\frac{r_1}{r_2}$=sin 15°	$\frac{r_1}{r_2}$=sin 30°	$\frac{r_1}{r_2}$=sin 50°	$\frac{r_1}{r_2}$ tends to unity
$\frac{S}{p}$.....	0.723	0.714	0.704	0.695	0.690	0.689
B.....	71°	67°	56°	44°	37°	32°
$\frac{r}{a}$.....	1	0.934	0.742	0.446	0.184	0
$\frac{z}{a}$.....	0	0.156	0.383	0.536	0.615	0.620

[1] The values, $\frac{1}{4}$ and 0.45 for μ, appearing in this paper are values selected by Love and Carothers respectively, in their computations.

The stresses at any point, Q, were computed by Carothers by expanding the various functions of w in zonal harmonics. Some of his computed values for $\frac{S}{p}$ are given in Table 2. These values were computed by taking $\mu=0.45$. It is seen from this table that the greatest value of s_{max} is near the perimeter of the loaded circular area where R=a and θ has values ranging from 80° to 90° (see Fig. 2). Love and Carothers are in good agreement as to the region of greatest shear.

TABLE 2

VALUES OF $\frac{S}{p}$ FOR DIFFERENT POSITIONS OF POINT Q, FIGURE 2, ACCORDING TO CAROTHERS, FOR $\mu=0.45$

θ	$\frac{S}{p}$ for R=a	$\frac{S}{p}$ for R=$\frac{2a}{3}$
0°	0.54	0.60
30°	0.55	0.60
45°	0.57	0.53
60°	0.58	0.50
75°	0.62	0.40
80°	0.63	0 30
85°	0.63	0.18
90°	0.63	0.05

POSSIBLE MEANS OF CHECKING THE VALUES OF LOVE AND CAROTHERS

The method of Love, although enormously complicated, is presented in great detail and it is possible to check his numerical values, using his method. Carothers, on the other hand, gives no clue as to what particular functions he expanded in zonal harmonics in making his numerical computations. It would seem desirable then to find some means of checking the values of both Carothers and Love without using exactly the same method of either of these two investigators. This is an exceedingly difficult undertaking for the reason that Love's solution is complete, all of the stresses, p_r, p_z and s_{rz}, being found throughout the entire region within a radial distance, a, of the loaded area. On the basis solely of his numerical values and without regard to how they were obtained, the same is true of Carothers' solution.

For the case of a uniform pressure, p, on a circular area it is easy to show (see Timoshenko (8), page 337) that for any point on the axis of symmetry and for $\mu=\frac{1}{4}$,

$$\frac{S}{p}=\tfrac{1}{4}+\tfrac{1}{4}\left[\frac{5a^2z-z^3}{(a^2+z^2)^{\frac{3}{2}}}\right] \quad \ldots\ldots 7$$

This expression may be expanded by the binomial theorem to give the infinite series,

$$\frac{S}{p}=\tfrac{1}{4}+\tfrac{5}{4}\left[\frac{z}{a}-\frac{1\times3}{2}\frac{z^3}{a^3}+\frac{1\times3\times5}{2\times4}\frac{z^5}{a^5}-\frac{1\times3\times5\times7}{2\times4\times6}\frac{z^7}{a^7}+,-\right]$$
$$-\tfrac{1}{4}\left[\frac{z^3}{a^3}-\frac{1\times3}{2}\frac{z^5}{a^5}+\frac{1\times3\times5}{2\times4}\frac{z^7}{a^7}+,-,\right] \quad \ldots\ldots 8$$

if a is greater than z, and

$$\frac{S}{p}=\tfrac{1}{4}+\tfrac{5}{4}\left[\frac{a^2}{z^2}-\frac{1\times3}{2}\frac{a^4}{z^4}+\frac{1\times3\times5}{2\times4}\frac{a^6}{z^6}-\frac{1\times3\times5\times7}{2\times4\times6}\frac{a^8}{z^8}+\right]$$
$$-\tfrac{1}{4}\left[1-\frac{1\times3}{2}\frac{a^2}{z^2}+\frac{1\times3\times5}{2\times4}\frac{a^4}{z^4}-,+,\right] \quad \ldots\ldots 9$$

if z is greater than a.

From a mathematical standpoint, it is not correct to expand a function in zonal harmonics that does not satisfy Laplace's equation. In potential theory, a Newtonian potential function, known to be harmonic (satisfies Laplace's equation) may first be computed on an axis

of symmetry and thereafter it may be computed at any point not on the axis, by expanding in zonal harmonics or Legendrian polynomials.

In general, the expression for a stress is a tensor (13) and is not therefore harmonic. However, the expression for a stress may consist of several parts, one or more of which may be harmonic, and it is legitimate to evaluate these by expanding in zonal harmonics. This, no doubt, was Carothers' procedure although he does not indicate it.

In the present consideration, the circle is not one of complete symmetry as Love has shown and in the absence of any general expression for $\frac{S}{p}$, known to be harmonic or otherwise, there is no sound mathematical basis for the following procedure which is to *assume* that the general expression for $\frac{S}{p}$ is harmonic and to pass from equations 7 and 8 to series of Legendrian polynomials. It is also assumed that the circle is symmetrical and this is not quite true.

If now the loaded circular area is considered as analogous to an electrically charged disk in potential theory, then one could substitute R for z in equations 8 and 9 and introduce Legendrian coefficients (14), thereby expanding equation 7 in zonal harmonics. When this is done, equation 8 becomes

neglecting all powers of $\frac{R}{a}$ beyond the sixth.

The terms, $P_1(\cos\theta)$, $P_3(\cos\theta)$, etc., are the Legendrian coefficients and numerical values for these coefficients, corresponding to different values of θ, Figure 2, may be found in various mathematical treatises. For R less than a and $\theta=0$, equation 10 becomes equation 8 and for R greater than a and $\theta=0$, equation 11 reduces to 9.

It should be emphasized now that aside from the geometric similarity, that is, a circle and an axis of symmetry in both cases, the problem of potential at a point due to a charged disk and the problem of shearing stresses beneath a loaded circular area have nothing in common.

In Table 1, for $\frac{r}{a}=0.184$, $B=37°$ and $\frac{r_1}{r_2}=\sin 50°$, it is found that $\theta=16°$, 40′ and $R=0.6416\,a$. The coefficients, $P_1(\cos\theta)$, $P_3(\cos\theta)$ corresponding to $\theta=16°$, 40′ are

$$P_1(\cos\theta)=0.9579$$
$$P_3(\cos\theta)=0.7609$$
$$P_5(\cos\theta)=0.4573$$
$$P_7(\cos\theta)=0.1207$$

Substituting these values in equation 10,

$$\frac{S}{p}=\tfrac{1}{4}+\tfrac{5}{4}(0.6141-0.3015+0.0930-0.0118)$$
$$-\tfrac{1}{4}(0.2010-0.0744+0.0101)=0.709,$$

which compares favorably with Love's value in Table 1 which is 0.690.

$$\frac{S}{p}=\tfrac{1}{4}+\tfrac{5}{4}\left[\frac{R}{a}P_1(\cos\theta)-\frac{1\times3}{2}\frac{R^3}{a^3}P_3(\cos\theta)+\frac{1\times3\times5}{2\times4}\frac{R^5}{a^5}P_5(\cos\theta)\right.$$
$$\left.-\frac{1\times3\times5\times7}{2\times4\times6}\frac{R^7}{a^7}P_7(\cos\theta)\right]-\tfrac{1}{4}\left[\frac{R^3}{a^3}P_3(\cos\theta)\right.$$
$$\left.-\frac{1\times3}{2}\frac{R^5}{a^5}P_5(\cos\theta)+\frac{1\times3\times5}{2\times4}\frac{R^7}{a^7}P_7(\cos\theta)\right]\ldots\ldots10,$$

if we neglect all powers of $\frac{R}{a}$ beyond the seventh, and equation 9 becomes

$$\frac{S}{p}=\tfrac{1}{4}+\tfrac{5}{4}\left[\frac{a^2}{R^2}P_1(\cos\theta)-\frac{1\times3}{2}\frac{a^4}{R^4}P_3(\cos\theta)+\frac{1\times3\times5}{2\times4}\frac{a^6}{R^6}P_5(\cos\theta)\right]$$
$$-\tfrac{1}{4}\left[1-\frac{1\times3}{2}\frac{a^2}{R^2}P_1(\cos\theta)-\frac{1\times3\times5}{2\times4}\frac{a^4}{R^4}(\cos\theta)\right.$$
$$\left.+\frac{1\times3\times5\times7}{2\times4\times6}\frac{a^6}{R^6}P_5(\cos\theta)\right]\ldots\ldots11,$$

The series in this case converges rapidly. By using Love's numerical values for p_r, p_z and s_{rz} ($\mu=\frac{1}{4}$) and equation 6 one finds that for $\frac{R}{a}=0.56$ and $\theta=45°$ that $\frac{S}{p}=0.66$. However, by equation 10 the corresponding value for $\frac{S}{p}$ is 0.75 which is 14 percent higher than Love's value. Again, for $\frac{R}{a}=0.19$ and $\theta=45°$, from Love's tables and equation 6, $\frac{S}{p}=0.42$ according to Love, which is exactly the same as the value obtained from equation 10. Jürgenson (15) has computed various values for $\frac{S}{p}$, using Carothers' tables for p_r, p_z, and s_{rz} and finds for example, that for $R=2a$ and $\theta=45°$, $\frac{S}{p}=0.25$. The corresponding valuc found by an expression similar to equation 11 is 0.27. Love's value for $\frac{S}{p}$ for $\frac{r}{a}=0.446$ and $\frac{z}{a}=0.536$ is 0.695 (Table 1), whereas equation 10 gives a value of 0.716 which is close to Love's values. For θ equal to 90° and $\frac{R}{a}$ less than one, the value of $\frac{S}{p}$ is 0.25 by equation 10, which is Love's value in this region. Equations 10 and 11 do not fit the boundary condition for $\theta=90°$ and R greater than a.

In all other cases it may be said that for θ greater than 45° and for $\frac{R}{a}$ or $\frac{a}{R}$ greater than $\frac{2}{3}$, the values for $\frac{S}{p}$ computed from equations 10 and 11 deviate by at least 15 percent (in many cases much more) from Love's numerical values. For all other positions of the point Q (Fig. 2) the agreement is generally within 15 percent. Carothers states that for values of $\frac{R}{a}$ or $\frac{a}{R}$ approaching unity, his functions expanded in zonal harmonics were "unsatisfactory" and he gives no clue as to an alternative procedure in this case. In the above computations the same limitations are realized.

Equations 10 and 11 have very definite limitations although they tend to give values that are in approximate agreement with those of Love and Carothers at various points.

Functions of solid angles are for the most part of interest only to the mathematician. A practicing engineer has more confidence in numerical values if he can check them by a simple graphical method.

The following simple device is suggested by the author for this purpose. Its use is very limited. The method is as follows:

It is desired, for example, to know the value of $\frac{S}{p}$ at the point where $\theta=45°$ and $R=\frac{2a}{3}$, the value of μ being taken as 0.45 (Carothers' assumed value). In Figure 3, AB is the diameter of the loaded circular area on the horizontal surface and OZ is its axis of symmetry. Draw the radial lines AQ, QB and OQ=R. Draw the line MN through O making an angle θ with AB. The projection of AB on MN is A'B', the minor axis of an ellipse. This ellipse is in the plane that is passed through MN perpendicularly to the plane of the paper. Its major axis is equal to AB which passes through the point O and is perpendicular to MN.

The length A'O is $a\cos\theta=\frac{1}{2}$ the minor axis and $a=AO$ is $\frac{1}{2}$ the major axis. Then the area of the ellipse formed by projecting the circle in the horizontal plane on the plane MN is, $\pi a\,(a\cos\theta)=\pi a^2\cos\theta$.

Let b=the radius of the equivalent circle, that is, the radius of the circle

having an area of $\pi a^2 \cos\theta$. Then $\pi b^2 = \pi a^2 \cos\theta$, or $b = a\sqrt{\cos\theta}$. Now replace the ellipse with the circle of radius $= b = a\sqrt{\cos\theta}$. A uniform pressure, p, perpendicular to the plane MN and distributed over the circle of radius $a\sqrt{\cos\theta}$ will produce approximately the same stress, s_{max}, at Q as is caused by the same

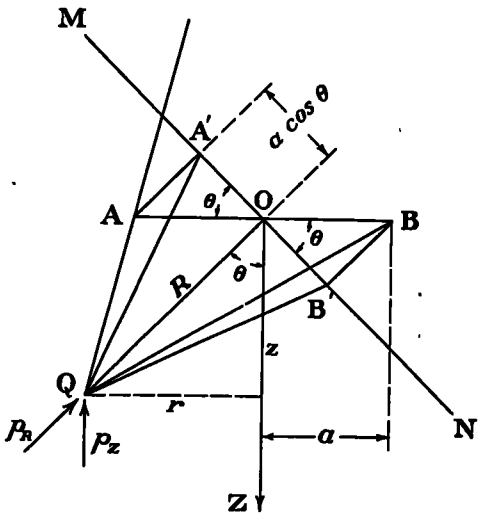


Figure 3. The Projection of the Loaded Circular Area, AB in the Horizontal plane on the plane MN is an ellipse with minor axis A'B' with major axis AB.

pressure, p, distributed over the circular area, AB, in the horizontal plane, provided θ is less than 45°. For $\theta = 0$, it is observed that p_R becomes p_z.

For the general case, the value of s_{max} when the point Q is on the axis of symmetry is obtained from the expression,

$$s_{max} = \frac{p}{2}\left[\frac{1-2\mu}{2} + (1+\mu)\frac{R}{(b^2+R^2)^{\frac{1}{2}}} - \frac{3}{2}\frac{R^3}{(b^2+R^2)^{\frac{5}{2}}}\right] \quad \ldots\ldots\ldots\ldots 12$$

where $b = a\sqrt{\cos\theta}$ = radius of the equivalent circle. For $R = \frac{3}{4}a$, $\theta = 45°$, $\cos\theta = \frac{1}{\sqrt{2}}$ and $\mu = 0.45$,

$$s_{max} \text{ at point } Q = \frac{p}{2}\left[0.05 + \frac{2\times 1.45}{3\left(\frac{1}{\sqrt{2}}+\frac{4}{9}\right)^{\frac{1}{2}}} - \frac{\frac{3}{2}\times\frac{8}{27}}{\left(\frac{1}{\sqrt{2}}+\frac{4}{9}\right)^{\frac{5}{2}}}\right] = 0.296p$$

Then $\frac{S}{p} = \frac{2s_{max}}{p} = 0.592$. Carothers obtains the value, $\frac{S}{p} = 0.53$ for this point (Table 2). The divergence from Carothers' value in this case is likely indicative of the error involved in transforming the elliptic area into one that is circular.

In addition to approximate methods, such as the foregoing, there are other means of checking precise numerical values, such as those published by Love and Carothers. One of such means is the photoelastic method. However, at present there has been but little progress in the analyses of three-dimensional problems of stress distribution by photoelastic devices. R. Weller (16) has suggested that it is possible to use the polarization caused by the scattering of light within a "cloudy" or opaque model in place of the usual analyzer in photoelastic investigations and that this method of polarization enables one to make analyses of three-dimensional stress systems very conveniently. An older method was to cool the model under load from an elevated to room temperature and then analyze it subsequently after slicing it into plates. This is an enormously complicated procedure.

D. P. Krynine (17) has described his device, called a stereogoniometer, which offers much promise in such studies and which has the advantages of simplicity and low cost. This device makes use of the principle of projecting areas from a plane to a spherical surface rather than from one plane to another plane as was done in the preceding example. By this procedure, the stress, p_z, is found in terms of the area of the projection of the loaded surface on the sphere and the volume of

the space bounded by this projected area, its projection in turn on the horizontal plane including the load and the perpendicular lines joining the perimeters of the two projected areas. The solid angle at a point below the loaded area is equal to the projected area on the sphere divided by the square of the radius of the sphere.

It is possible to compute the solid angle w, without this device, as follows: On the axis of symmetry, the expression for the solid angle, w is

$$w=2\pi\left(1-\frac{z}{R}\right)\ldots\ldots\ldots 13$$

where $R=(a^2+z^2)^{\frac{1}{2}}$. The general expression for w (see equation 3) is harmonic and it is mathematically legitimate to expand it in zonal harmonics. By so doing, one obtains the two expressions,

$$w=2\pi-2\pi\left[\frac{R}{a}P_1(\cos\theta)-\tfrac{1}{2}\left(\frac{R}{a}\right)^3 P_3(\cos\theta)+\tfrac{3}{8}\left(\frac{R}{a}\right)^5 P_5(\cos\theta)-\tfrac{15}{48}\left(\frac{R}{a}\right)^7 P_7(\cos\theta)+,-,\text{etc.}\right]\ldots\ldots 14$$

for R less than a, and

$$w=2\pi-2\pi\left[1-\tfrac{1}{2}\left(\frac{a}{R}\right)^2 P_1(\cos\theta)+\tfrac{3}{8}\left(\frac{a}{R}\right)^4 P_3(\cos\theta)-\tfrac{15}{48}\left(\frac{a}{R}\right)^6 P_5(\cos\theta)+\tfrac{105}{384}\left(\frac{a}{R}\right)^8 P_7(\cos\theta)+,-\text{etc.}\right]\ldots\ldots 15$$

for R greater than a.

For $\frac{R}{a}=\frac{2}{3}$ and $\theta=45°$, one obtains $w=1.07\pi$ from equation 14. Then substituting this value in the expression which is valid only for $\mu=\frac{1}{2}$,

$$p_z=\frac{3p}{2\pi}w\cos^2\theta\ldots\ldots\ldots 16$$

it is found that $p_z=0.803p$. The value for p_z at $R=\frac{2}{3}a$ and $\theta=45°$ is 0.858p when $\mu=0.45$, according to Carothers.

Similarly, for $R=2a$ and $\theta=45°$, $w=0.195\pi$ from equation 15 and on substitution in equation 16, $p_z=0.146p$. The corresponding value obtained by Carothers for $\mu=0.45$ is 0.158p. The differential of equation 16 was proposed by D. P. Krynine (17). The stereogoniometer is a device that performs an exact mechanical integration of the differential of equation 16, θ and w being both variable in the integration.

Equation 16 does not satisfy the boundary conditions, $\theta=0$ and $\theta=90°$. For a uniformly loaded circular area it gives results which are approximately correct for θ greater than 30° and less than 60°, R being greater than zero. On the axis of symmetry,

$$p_z=p\left[1-\frac{z^3}{(a^2+z^2)^{\frac{3}{2}}}\right]\ldots\ldots 17$$

If θ is taken as the average angle between the vertical direction and the radial distance R drawn from any element of loaded surface to the point Q, considering all of the elements of the loaded area, the computations by equation 16 become exact. The analytical procedure of obtaining this average θ is a very difficult one. For R greater than $\frac{3}{2}a$, equation 16 together with equations 14, 15 and 17 give values for p_z that do not diverge more than 15 percent (in most cases much less) from Carothers' values when θ does not exceed 60°.

The purpose of the foregoing procedure is to indicate that in substance, at least, the mechanical method used by Krynine is correct. Its outstanding advantage is that in its use the evaluation of the stress, p_z, can be accomplished for any contour of uniformly loaded area. It is not necessary that there be axial symmetry.

The precise analytical method of obtaining p_z at a point under a uniformly loaded circular area is indicated by equation 5 which is Carothers expression for this stress. Since

$$w = \int\int \frac{z}{R_1^3}\, dx\, dy,$$

Then

$$\frac{\partial w}{\partial z} = \int\int \left(\frac{1}{R_1^3} - \frac{3z^2}{R_1^5}\right) dx\, dy \quad \ldots\ldots 18$$

and

$$z\frac{\partial w}{\partial z} = \int\int \frac{z}{R_1^3}\, dx\, dy - 3\int\int \frac{z^2}{R_1^4}\left(\frac{z}{R_1^3}\right) dx\, dy \quad \ldots\ldots 19$$

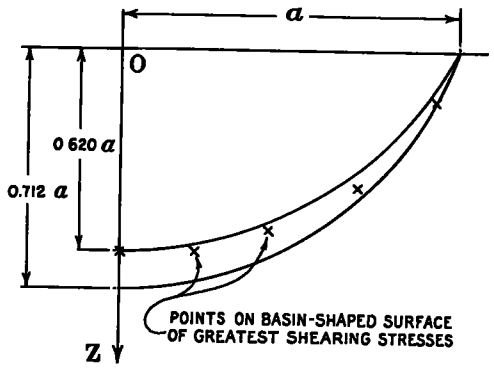


Figure 4. Points at which the Stressed Material under a uniformly Loaded Circular Area Would be Most Likely to Fail.

It is possible to evaluate these two integrals at any point Q. The first integral is w, known to be harmonic, and it may be computed at any desired point by means of equations 14 and 15. The second integral may be simplified by expanding $\frac{z}{R_1^3}$ in Legendrian polynomials since $\nabla^2\left(\frac{z}{R_1^3}\right) = 0$. With w and $z\frac{\partial w}{\partial z}$ thus evaluated, the stress, p_z, at any point, Q, is computed from equation 5.

For a uniform pressure produced by a wheel load on the nonrigid surface, the distribution of vertical pressure, p_z, over the area of subgrade bounded by the radial line R is far from uniform as Hawthorne (18) assumed in his analysis.

Love has shown that for any given value of μ, there is a certain value of R, expressed as some multiple of a, for which s_{max} has a greatest value. Thus for $\theta = 45°$, the greatest value of s_{max} on the radial line is at any point where R = 0.73a. Similarly for $\theta = 90°$, the greatest value of s_{max} is at points where R = a, that is, at points just under the perimeter. If the points of "partial maxima stress difference" on all of the radial lines for all values of θ are connected, the locus of such points is found to be a "basin-shaped surface of revolution about the axis of the circle." It passes through the circle and lies between two segments of spheres, which have their centers on the axis of symmetry and pass through the circle (see Fig. 4). These two spheres cut the axis of symmetry at depths equal to 0.620a and at 0.712a when $\mu = \frac{1}{4}$. According to Love, it is reasonable to conclude that the foundation under a round pillar would be most likely to give way at points on such a basin-shaped surface and that it would be nearly as likely to give way at one point of this surface as at any other. The values given in Table 1 are for points on the basin-shaped surface.

A practical consideration is the following: If the supporting subgrade is of questionable supporting power, such as medium or soft clay, then considering all possible values of μ, and the uncertainty of the effect of contact area where pavement and subgrade meet, it would be on the side of safety to have the thickness

of the more resistant flexible pavement at least equal to a, the radius of the equivalent circle. This precaution would tend to confine the dangerous surface to the surfacing material. But it is again necessary to emphasize the fact that since the flexible pavement and the subgrade are very different materials, all such conclusions must be considered as only generally indicative and not strictly true in a quantitative sense.

The greatest values of s_{max} may be computed on the axis of symmetry from equation 12 for different values of μ. Table 3 contains such values and the points at which they are found are defined in terms of $\frac{z}{a}$. It is interesting to note from this table that the shearing stress does not vary greatly as μ varies from 0.25 to 0.50.

TABLE 3

MAXIMUM VALUES OF $\frac{s_{MAX}}{p}$ ON THE AXIS OF SYMMETRY FOR DIFFERENT VALUES OF

μ	$\frac{s_{max}}{p}$	$\frac{z}{a}$ at point of s_{max}
0.25	0 34	0.62
0.30	0.33	0.64
0.40	0.31	0.67
0.45	0.30	0.69
0.50	0.29	0.71

Benkelman (4) has rightfully emphasized our lack of information concerning conditions at the boundary plane of contact between the flexible surface and the supporting medium. It is possible that this information may be obtained by intelligent and careful experimentation, making use of adequate theory in all such experimental procedures.

CONCLUSIONS

On the basis of Love's complete solution of the problem of stresses under a uniformly loaded circular area, it is indicated that the greatest shearing stresses may be confined within the flexible pavement and not reach the subgrade if the thickness of the flexible pavement is no less than the radius of a circle having an area equal to the plane of contact between pneumatic tire and flexible pavement.

The conclusions reached by Carothers are in good agreement with those reached independently by Love, but the method used by Carothers requires a more complete presentation.

In the case of a uniform load on a circular area the variation in shearing stress, as Poisson's ratio varies from 0.25 to 0.50, is relatively small.

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SHEARING STRESSES AND SURFACE DEFLECTIONS DUE TO TRAPEZOIDAL LOADS

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SYNOPSIS

In this paper the stresses and surface deflections of a semi-infinite elastic medium, and of an elastic layer of finite depth are obtained by the principle of superposition. When these effects are known for a concentrated surface load, the corresponding results for distributed or continuous surface loads may be obtained by summation or integration methods.

In the first part of the paper the author determines all the stresses in the supporting medium for surface loads which simulate triangular, symmetrical and unsymmetrical trapezoidal surface loads due to road fills, and determines the maximum shearing stresses under such loads. In particular it is shown that the depth at which this maximum shear occurs under a symmetrical trapezoidal fill is at a depth approximately equal to one-half of the length of an equivalent rectangular load of equal intensity. This maximum shear value does not exceed 0.318 p where p is the maximum intensity of the surface loading.

In the second part of the paper, superposition methods are employed to determine the stresses and surface deflection of a finite elastic layer supported by a rough rigid base, and bearing a trapezoidal surface load. It is shown that the stresses are no longer independent of Poisson's ratio ν, and that shearing failure occurs within the layer before it occurs at the supporting base. The surface deflections are evaluated (using $\nu = 0.25$) for several types of surface loading and varying depths of the elastic layer. It is also shown that the results of the first part of the paper can be derived as a limiting case by allowing the depth of the layer to become infinite.

It is recognized that these solutions furnish basic pictures, departures from which in the imperfectly elastic material must be examined and judged by experimental progress.

It is a common observation that the theory of elasticity is useful beyond the range of only those media which, in the true sense, are purely elastic. Thus, soil mechanics, which deals with materials that are far from being perfectly elastic has profited from the results of classical theory for the transmission of surface pressures in a supporting medium. These solutions only furnish basic pictures, departures from which in the imperfectly elastic material must be examined and judged by experimental progress.

In this paper the analytical method of Boussinesq, assuming plane strain conditions, is applied to two dimensional problems for the determination of the distribution of the stresses and the surface displacements of a semi-infinite elastic medium bearing surface loads. By a plane strain or plane deformation problem is meant a stressed body which is very long in one direction in comparison with the other transverse dimensions, so that the stresses and deformations are the same in all transverse sections. It is also assumed that the supporting medium has uniform elastic properties, and that the resulting distribution is due to the surface loads and not to the body forces of the supporting medium.

In the paper, two cases are considered: Case (a), The supporting medium is infinitely deep and has a horizontal surface plane,[1] Case (b), The medium is an elastic layer of finite depth h with a horizontal surface plane, and resting on a rough rigid base.

[1] In the appendix the case of an inclined surface plane is treated. These results were appended at the suggestion of Dr. Karl Terzaghi whose written discussion was presented to the writer.

In both cases the chief consideration is to ascertain (1) the distribution of stresses and (2) the surface deflections. In particular the author determines the principal shearing stresses under symmetrical surface loads whose intensity is indicated by the trapezoidal loadings of Figures 2 and 8. If one may assume that this loading simulates a road fill on a naturally horizontal earthy medium, then the results may indicate the nature of the stress distribution. No assertion is made that the results hold in such an imperfectly elastic medium. The analysis is more likely to hold for the stress distribution than for the surface deflections.

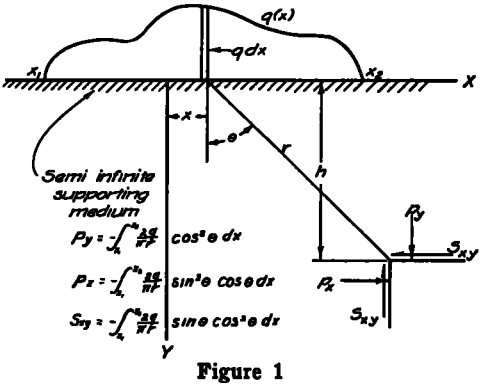


Figure 1

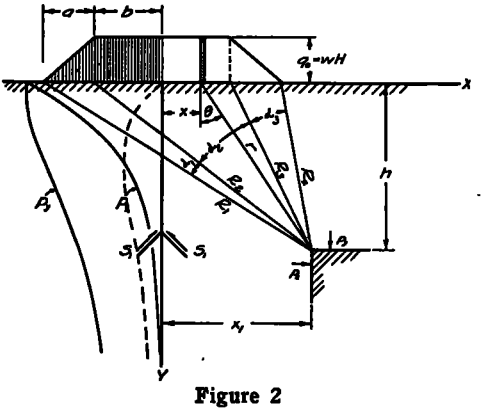


Figure 2

STRESSES IN A SEMI-INFINITE MEDIUM

Consider a case of plane deformation where a loading q(x) in Figure 1 produces stresses and deformations in the supporting medium. From the elastic theory of Boussinesq one deduces the stresses p_x, p_y, and S_{xy} from the following integrals (the coordinates and elements are shown in Fig. 1).

$$p_x = -\int_{x_1}^{x_2} \frac{2q(x)}{\pi r} \sin^2\theta \cos\theta \, dx,$$

$$(1) \qquad p_y = -\int_{x_1}^{x_2} \frac{2q(x)}{\pi r} \cos^3\theta \, dx,$$

$$S_{xy} = -\int_{x_1}^{x_2} \frac{2q(x)}{\pi r} \cos^2\theta \sin\theta \, dx.$$

In Eqs. (1), the integrands are the stresses for an elemental loading q dx. By employing summation or integration for discrete or continuous loads, the stresses on elements at $x = x_1$ and at depth $y = h$ may be obtained. The results for the symmetrical trapezoidal loading of Figure 2 have been given by Palmer.[2] Slightly more general results are given for the case of Figure 3a where the central trapezoidal loading has two additional triangular loads of the same slope. These stresses are

$$p_y = -\frac{q_0}{\pi}\left[\left(\alpha_1 + \frac{\alpha_2}{2}\right) + \frac{b}{a}(\alpha_1 + \alpha_3) + \frac{x_1}{a}(\alpha_1 - \alpha_3) - \frac{x_1\alpha_2}{2b}\right] - \frac{q_1}{\pi}\left(\alpha_3 + \frac{\alpha_2}{2} + \frac{x_1\alpha_2}{2b}\right),$$

$$(2) \quad p_x = p_y - \frac{2q_0h}{\pi a} \log\frac{R_2R_3}{R_1R_4} - \frac{(q_1 - q_0)h}{\pi b} \log\frac{R_3}{R_2},$$

$$S_{xy} = \frac{hq_0}{\pi a}(\alpha_1 - \alpha_3) + \frac{(q_1 - q_0)h\alpha_2}{2\pi b},$$

[2] Palmer, L. A., "Principles of Soil Mechanics Involved in the Design of Retaining Walls and Bridge Abutments," *Public Roads*, Vol. 19, pp. 193-207, 1938.

where a_1, a_2, and a_3 are the angles subtended at the point in the supporting medium by the loading. The constants q_0 and q_1 are indicated intensities of the loading. The distances R_1, R_2, R_3, and R_4 in Figure 3a correspond to similar distances shown in Figure 2.

shows that there is an equal compressive stress or hydrostatic pressure directly under a normal surface loading. Another important observation is that all the stresses are independent of the elastic moduli of the supporting medium. Neither of these last two statements hold

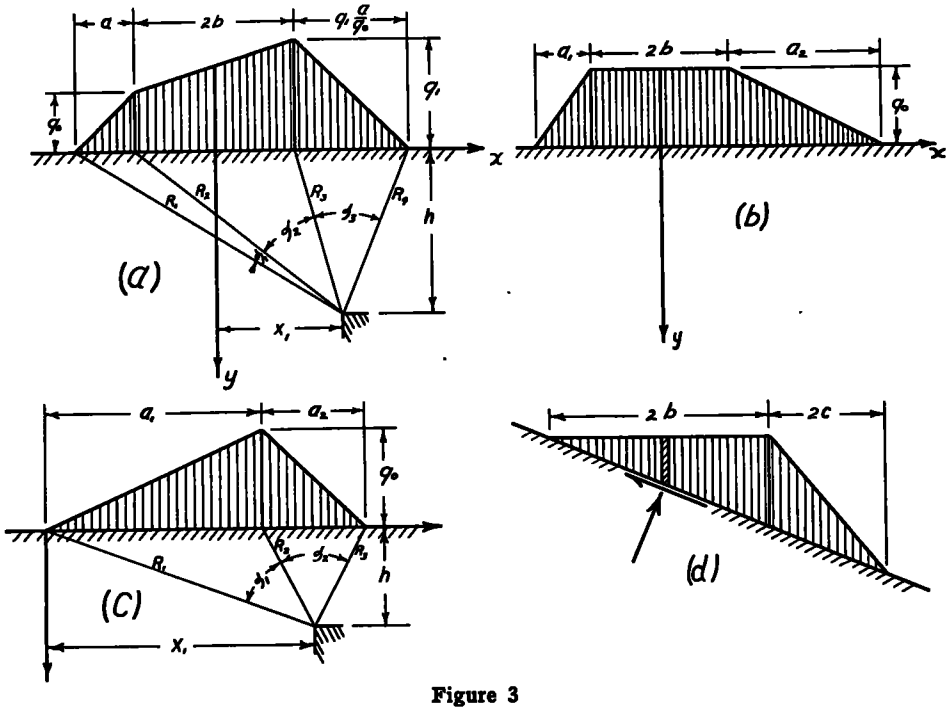


Figure 3

An examination of Eqs. (2) shows that when $h=0$, the shearing stress $S_{xy}=0$, that is, there is no tangential loading on the top surface. Also $p_x=p_y$ when $h=0$, for the finite layer problem treated in the latter part of this paper.

For the unsymmetrical trapezoidal loading of Figure 3b, these stresses are

$$p_y=-\frac{q_0}{\pi}\left[(a_1+a_2+a_3)+\frac{b}{a_1}(a_1+ka_3)+\frac{x_1}{a_1}(a_1-ka_3)\right],$$

$$p_x=p_y+\frac{2q_0}{\pi}\left[\frac{h}{a_2}\log R_4/R_3-\frac{h}{a_1}\log R_1/R_2\right], \tag{3}$$

$$S_{xy}=\frac{q_0 h}{\pi a_1}(a_1-ka_3),$$

and outside the loading area, where $a_1=a_2=a_3=0$, $p_x=p_y=0$. Under the loading $p_x=p_y=$unit load applied. This where the constant $k=a_1/a_2$ is the ratio of the projections of the sloping sections. The comments concerning Eqs. (2) hold

for Eqs. (3). If $k=1$ in Eqs. (3) or if $q_1=q_0$ in Eqs. (2) the stresses reduce to the results given by Palmer for a symmetrical trapezoidal load. For the triangular loading of Figure 3c the results may be obtained from Eqs. (3) by letting b approach zero, $a_2=0$ and $R_2=R_3$, that is (with a_3 becoming a_2),

$$
\begin{aligned}
p_y &= -\frac{q_0}{\pi}\left[\frac{a_1 x_1}{a_1} + a_2(1+a_1/a_2-x_1/a_2)\right],\\
p_x &= p_y + \frac{q_0}{\pi}\left[\frac{2h}{a_1}\log R_2/R_1 - \frac{2h}{a_2}\log R_3/R_2\right], \qquad (4)\\
S_{xy} &= \frac{q_0 h}{\pi}(a_1/a_1 - a_2/a_2).
\end{aligned}
$$

Many additional special cases could be obtained such as a rectangular loading (by allowing $a_1=a_2=0$ in Eqs. (3)), or a trapezoidal loading such as the central portion of Figure 3a (by allowing $a=0$ in Eqs. (2)), or an infinitely long embankment loading when $a_2=\infty$ in Figure 3c. All the above results have been derived by superposition methods upon the assumption that the loads exert only a normal pressure upon the supporting medium. Figure 3d indicates a case where the load is supported by a medium whose surface is not perpendicular to the gravitational field which is taken to be vertical. In this case the surface loading exerts a normal as well as a tangential stress upon the inclined surface of the supporting medium. The stresses for this type of loading are included in the appendix.

MAXIMUM SHEARING STRESSES

Again considering the special case of Figure 2, one finds the greatest shearing stresses occur on the vertical axis under the symmetrical loading. Since the shearing stress S_{xy} vanishes on this axis, the normal stresses p_x and p_y are principal stresses (these curves are shown with a lateral scale which has a value of q_0 at the surface). Hence on elements inclined 45° with this vertical axis, the principal shearing stress S_1 is one-half the difference of these normal stresses (Fig. 2 shows this S_1 stress by a broken line),

$$S_1 = \frac{p_y - p_x}{2} = \frac{2q_0 h}{\pi a}\log R_1/R_2 = \frac{q_0 h}{\pi a}\log\frac{h^2+(a+b)^2}{h^2+b^2}. \qquad (5)$$

The depth $\bar{h}$, at which this shearing stress becomes a maximum depends on the dimensions a and b of the loading. This value of $\bar{h}$ is a solution of the equation

$$\frac{2h^2(a^2+2ab)}{(h^2+b^2)[h^2+(a+b)^2]} = \log\frac{h^2+(a+b)^2}{h^2+b^2}.$$

Some special values are as follows:

When:		Then:		
	$a=0.0b$		$\bar{h}=1.00b$,	$S_1=0.318q_0=q_0/\pi$
	$a=0.5b$		$\bar{h}=1.24b$,	$S_1=0.316q_0$
	$a=1.0b$		$\bar{h}=1.48b$,	$S_1=0.314q_0$
	$a=2.0b$		$\bar{h}=1.92b$,	$S_1=0.310q_0$.

If one may assume for the sake of an application of these results, that the loading of Figure 2 is an actual road fill of height H with the slope fills depending upon H, then if the loading intensity is proportional to the height, the critical depth $\bar{h}$ depends upon H as well as the dimensions of the loaded regions. In Tables 1 and 2 accurate values of $\bar{h}$ and

the corresponding S_1 stresses are given for a range of values of b and H, with variable sloping fills and differing densities of loading materials.

One observes that if a symmetrical trapezoidal loading is replaced by a rectangular load of length $2b+a$, then the critical depth is $\bar{h}=b+a/2$ and S_1 has a maximum value of $0.318q_0$. These values are only slightly greater than the corresponding values of Tables 1 and 2. Thus a reasonably safe field rule is to investigate the shearing capacity of the supporting layer at a depth approximately equal to half the length of equivalent rectangular load.

TABLE 1

VALUES OF $\bar{h}$ FOR VARIOUS ROADWAYS, SLOPES AND HEIGHTS OF FILL

H ft.	1.5 to 1 slope			1.65 to 1 slope			1.8 to 1 slope		
	Roadway =2b			Roadway =2b			Roadway =2b		
	32 ft.	36 ft.	40 ft.	32 ft.	36 ft.	40 ft.	32 ft.	36 ft.	40 ft.
10	23	25	27	24	26	28	24	26	28
20	30	32	34	31	33	35	33	35	37
30	37	39	41	39	41	43	41	43	45
40	44	46	48	46	48	50	49	51	53
50	50	52	54	54	56	58	57	59	61
60	57	59	61	62	64	66	66	68	70
70	64	66	68	69	71	73	74	76	78
80	71	73	75	77	79	81	82	84	86
90	78	80	82	84	86	88	91	93	95
100	85	87	89	92	94	96	99	101	103

TABLE 2

SHEAR STRESS IN LB. PER SQ. FT. FOR VARIOUS HEIGHTS OF FILL AND UNIT WEIGHT OF FILL MATERIAL

H ft.	110 lb. per cu. ft.	120 lb. per cu. ft.	130 lb. per cu. ft.
10	341	372	403
20	682	744	806
30	1,023	1,116	1,210
40	1,364	1,488	1,612
50	1,705	1,862	2,016
60	2,046	2,231	2,419
70	2,387	2,602	2,820
80	2,728	2,975	3,223
90	3,069	3,348	3,627
100	3,410	3,720	4,030

SURFACE DEFLECTION IN A SEMI-INFINITE MEDIUM

One may apply the superposition method to find the surface deflection on an elastic medium due to distributed loads if the deflection for a point load is known. From the classical elastic theory [3] only the relative deflection of this surface can be derived. Some point whose deflection is taken as zero must be chosen as a reference point, and all deflections are then referred to this point. For convenience it may be chosen as a point interior to the supporting medium, or upon the top surface. In this paper it is expedient to choose this reference point as some point on the top surface and external to the loaded area. Since this problem is one of plane displacement, it appears that a reference point in the surface admits of experimental observation.

[3] Timoshenko, S., "Theory of Elasticity," pp. 86-88. His case is the case of plane stress and all deflections are made relative to a reference point on the vertical axis at which the deflection vanishes.

In Figure 4 the surface deflection relative to a point d due to an elemental surface loading q dx is

$$(6)\quad w(x) = 2q\,dx\frac{(1-\nu^2)}{\pi E}\log\frac{d-x_1}{|x-x_1|},$$

where x_1 is the abscissa of the loading element, E the compression modulus and ν is the Poisson's ratio of the supporting

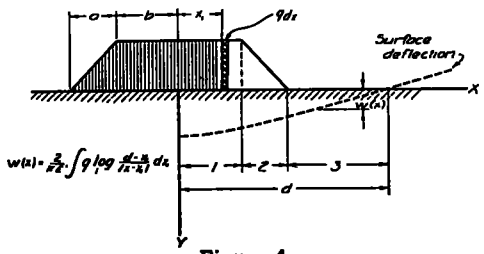


Figure 4

medium. The distance d is measured from the origin. It is a surface point at which the deflection vanishes and relative to which all interior deflections are measured. There is nothing in the analysis by which this value can be found. It is an indeterminate [4] and the results are not valid beyond this value of d.

For the trapezoidal loading, Figure 4 which has discontinuities in the slopes, the integration leads to three different forms of the deflection expressions. These may be denoted by w_1, w_2, and w_3 and are valid in the regions 1, 2, and 3 respectively of Figure 4. However all three may be written by a single expression if absolute values are used in the logarithmic terms.

$$(7)\quad w(x) = 2q/\pi E'\left[K + \frac{b^2+x^2}{2a}\log\frac{|x^2-b^2|}{|x^2-(a+b)^2|} + bx/a\log\frac{x+b}{|x-b|} + \frac{x(a+b)}{a}\log\frac{|a+b-x|}{a+b+x} + (b+a/2)\log\frac{d^2-(a+b)^2}{|x^2-(a+b)^2|}\right],$$

where

$$K = (b^2+d^2)/2a\log\frac{d^2-(a+b)^2}{d^2-b^2} + bd/a\log\frac{d-b}{d+b} + \frac{d(b+a)}{a}\log\frac{d+a+b}{d-a-b},$$

and

$$E' = E/(1-\nu^2).$$

The constant K depends only on the dimensions of the loaded region and the value of d. Each term of Eq. (7) contains a natural logarithm of a dimensionless ratio, and hence the bracketed expression is of the same dimension as b or a. One notes that the elastic moduli of the supporting medium appear only in the leading factor $2q/\pi E'$ which is dimensionless. Apart from this factor the elastic deflection of the top surface is not influenced by the type of the elastic supporting medium. Since the leading factor is purely a scale factor, it may be said to be a "foundation modulus" [5] and capable of experimental determination.

Equation (7) may be written out for each of the three regions and a continuous deflection curve drawn for selected ratios of d/a and d/b such as shown in Figures 5a and 5b. One notes that for a fixed value of d these curves do not cross each other except at the reference point d where the deflection vanishes. With increasing loading areas

[4] While d appears as an integration constant, it is not possible to assume that the deflection vanishes at infinity, for then all the displacements near the finite loading become infinite. M. A. Biot has indicated this in a plane stress problem in *Journal of Applied Mechanics*, Vol. 4, No. 1, pp. 1-7, 1937.

[5] In an elastic medium whose moduli are known, the quantity $E' = E/(1-\nu^2)$ is also known. Assuming the possibility of an application to a field experiment with the earth as a supporting medium, this foundation modulus and the value of d admit of an actual determination by two deflection readings. Even in an ideal medium, the value of d must be obtained by experiment.

(having a fixed maximum intensity q) the maximum deflection increases. This is more evident in Figure 6 where the maximum central deflection is shown for a range of ratios of a/b and d/b. From Eq. (7) this maximum deflection is

(8) $$w(0) = 2q/\pi E' \left[K + (2b+a)/2 \log \frac{d^2-(a+b)^2}{(a+b)^2} + b^2/a \log \frac{b}{a+b} \right].$$

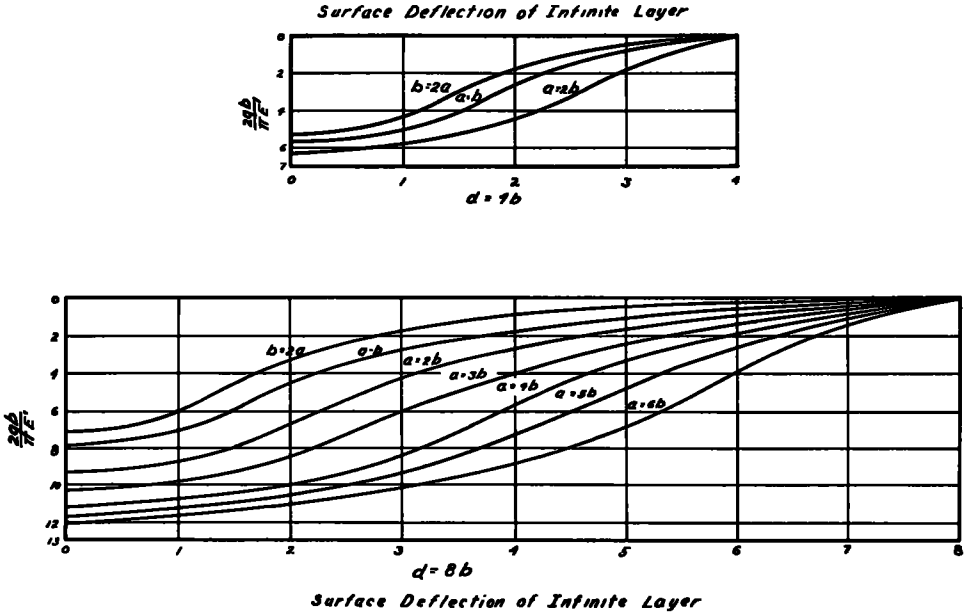


Figure 5

From Figure 6 one is lead to the conclusion that as d increases indefinitely the deflection in the finite portions of the plane also increases beyond bound. This fact will be referred to in the analysis of the problem of an elastic layer of finite depth.

STRESSES IN A FINITE ELASTIC LAYER

In Figure 7 is shown an elastic layer of finite depth h supported on a rough rigid base. The surface loading is F(x). Again the assumption is made that all transverse sections are under the same force systems and the problem is one of plane deformation. There is considerable literature available regarding this and related problems. Carothers[6] gave an incorrect solution for this case. Timoshenko objected to Carothers' solution by oral discussion at the time, but the error was not generally known until recently.[7] The first correct solution for the plane stress case was given by Marguerre.[8] It can be shown that the results of this

[6] Carothers, S. D., *Proceedings*, Int. Math. Congress, Vol. 2, Toronto, Canada, 1924.

[7] L. Jurgenson, *Journal*, Boston Society of Civil Engineers, Vol. 21, 1934 and in *Proceedings*, International Conference on Soil Mechanics, Vol. 2, 1936, derives formulas for other loadings by assuming Carothers' incorrect results. W. Passer, in Sitz. B. Akad. Wiss. Wien, Vol. 144, pp. 267, 1935, first gave the correct solution of the axially symmetric case on a rough base and refers to Carothers' error.

[8] Marguerre, K., in *Ing. Archiv*, Vol. 2, pp. 108-117 (1931) gives the plane stress case on a rough base, and in *Ing. Archiv*, Vol. 4, pp. 332-353 (1933) treats the axially symmetric case on a rough base or a yielding base.

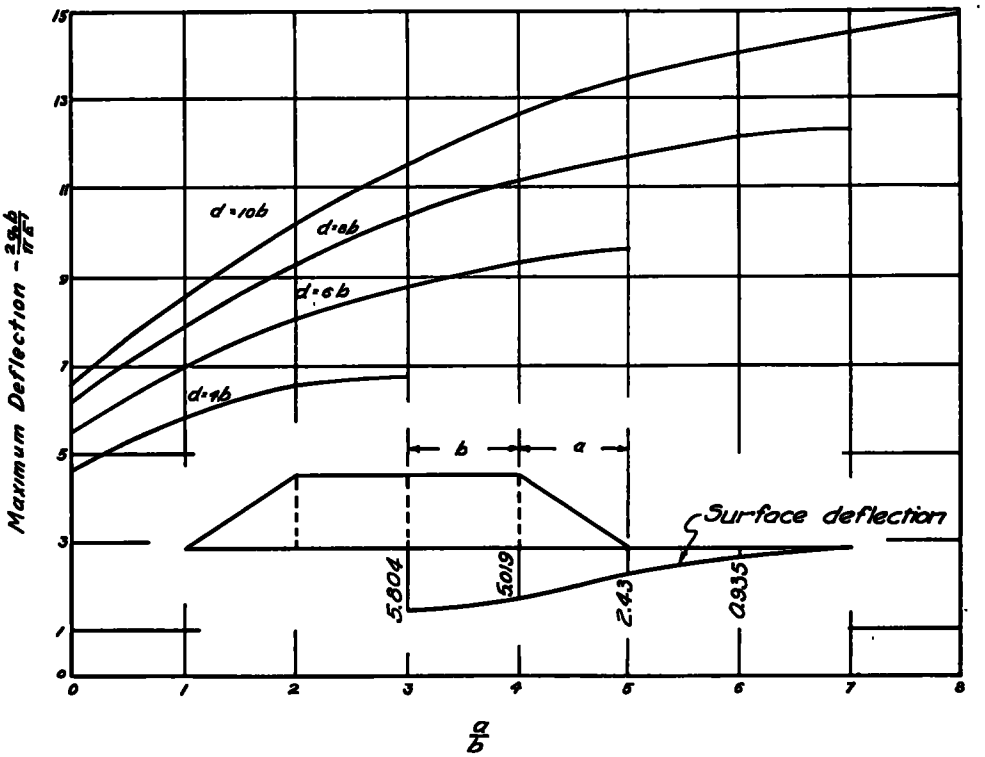


Figure 6

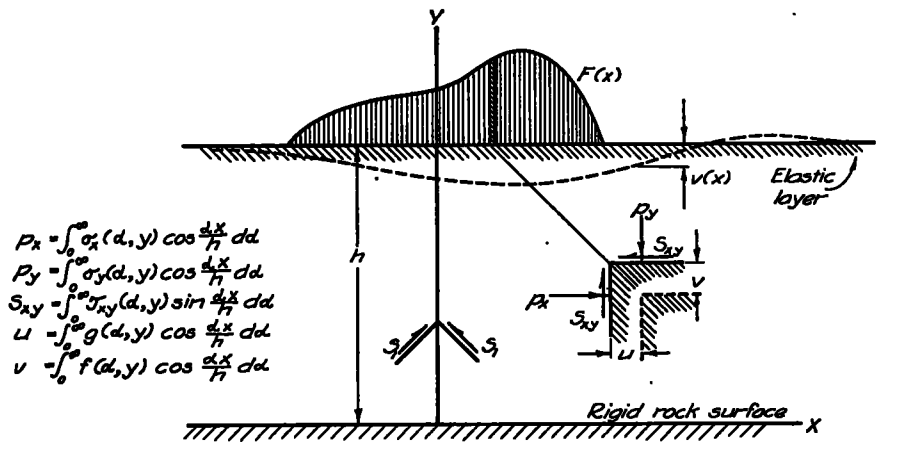


Figure 7

paper are derivable from his results by the usual transition from plane stress to plane deformation. Biot [9] has solved the problem correctly not only for plane deformation but also for the axially symmetric case for the special value of Poisson's ratio $\nu=0.5$. More recently and quite unknown to the writer, Pickett [10] has considered both the plane strain and the axially symmetric case. The immediate interest of the present paper was to consider the relationship of the maximum shearing stresses under the center of a symmetrical trapezoidal loading with the shearing stresses at the rough base, and to find the surface deflections.

Let the coordinate system be that of Figure 7. The conditions at the rock base where $y=0$ are that the horizontal and vertical displacements (u, v) shall vanish. At the surface $y=h$, the shear, $S_{xy}=0$ and p_y is the loading. Hence

$$(9) \qquad [u]_{y=0}=[v]_{y=0}=0;\ [S_{xy}]_{y=h}=0;\ [-p_y]_{y=h}=\text{loading}.$$

Now one seeks to find a stress function ϕ satisfying the biharmonic equation $\nabla^4\phi=0$ such that all the boundary conditions of Eqs. (9) are satisfied. For symmetrical normal surface loads which are finite in the neighborhood of the origin, a suitable stress function is

$$(10) \qquad \phi=\int_0^\infty [A\cosh ay+B\sinh ay+Cay\cosh ay+Day\sinh ay]\cos ax\,da$$

where A, B, C, and D are constants to be determined. One may formulate the boundary conditions in Eq. (9) by the relationships

$$p_x=\phi_{yy};\ p_y=\phi_{xx};\ S_{xy}=-\phi_{xy},$$

$$(11) \quad Eu=\int[(1-\nu^2)\phi_{yy}-\nu(1+\nu)\phi_{xx}]\,dx,$$

$$Ev=\int[(1-\nu^2)\phi_{xx}-\nu(1+\nu)\phi_{yy}]\,dy.$$

The first three conditions of Eqs. (9) lead to

$$B=(1-2\nu)C=\frac{A(1-2\nu)}{2(1-\nu)}\left[\frac{ah\cosh ah-(1-2\nu)\sinh ah}{ah\sinh ah+(1-\nu)2\cosh ah}\right] \qquad (12)$$

$$D=\frac{-A}{2(1-\nu)}.$$

To determine A, it is convenient to express the symmetric loading by the Fourier integral

$$F(x)=2/\pi\int_0^\infty\cos ax\,da\int_0^\infty F(\lambda)\cos a\lambda\,d\lambda$$

$$=\int_0^\infty Q(a)a^2\cos ax\,da$$

where

$$(13) \quad a^2Q(a)=2/\pi\int_0^\infty F(\lambda)\cos a\lambda\,d\lambda.$$

Now the last of Eqs. (9) and the definition of $p_y=\phi_{xx}$ lead to the condition

$$Q(a)=A\cosh ah+B\sinh ah+Cah\cosh ah+Dah\sinh ah$$

with B, C, and D given by Eqs. (12) and $Q(a)$ by Eq. (13) for any symmetrical loading. Accordingly one has

$$A=[2Q(a)/\Delta](1-\nu)[ah\sinh ah+2(1-\nu)\cosh ah], \qquad (14)$$

where

$$\Delta=(3-4\nu)\cosh^2 ah+(1-2\nu)^2+a^2h^2.$$

This completes the determination of the constants. Some special values of stresses p_x, p_y, and S_{xy} will be recorded. The general values of these stresses do

[9] Biot, M. A., *Physics*, Vol. 6, pp. 367-375 (1935).

[10] Pickett, G., in *Proceedings*, Highway Research Board, Vol. 2, pp. 35-48, 1938 has extended Biot's work for the rough rigid base. In private correspondence, Professor Pickett

not concern us at present, although it may be shown that they reduce to known results [11] when the value of h becomes infinite. In fact the limiting process shows that only in the infinite layer do the stresses become independent of Poisson's ratio.

The following are special values of the stresses.

(15)

$$p_x\Big|_{y=0} = -\int_0^\infty \frac{Q(a)}{\Delta}\,[2\nu a h \sinh ah + 4\nu(1-\nu)\cosh ah]\,a^2\cos ax\,da,$$

$$S_{xy}\Big|_{y=0} = -\int_0^\infty \frac{Q(a)}{\Delta}\,2(1-\nu)\,[ah\cosh ah - (1-2\nu)\sinh ah]\,a^2\sin ax\,da,$$

$$p_y\Big|_{y=0} = -\int_0^\infty \frac{Q(a)}{\Delta}\,[2(1-\nu)ah\sinh ah + 4(1-\nu)^2\cosh ah]\,a^2\cos ax\,da,$$

$$S_1\Big|_{x=0} = \int_0^\infty \frac{Q(a)}{\Delta}\Big\{(1-2\nu)\,[ah\sinh ah + 2(1-\nu)\cosh ah]\cosh ay$$
$$+2(1-\nu)\,[ah\cosh ah - (1-2\nu)\sinh ah]\sinh ay$$
$$+\,[ah\cosh ah - (1-2\nu)\sinh ah]\,ay\cosh ay$$
$$-\,[ah\sinh ah + 2(1-\nu)\cosh ah]\,ay\sinh ay\Big\}a^2\,da.$$

These equations give the stresses at the rough base and the principal shearing stress under the center of a symmetrical loading. In order to evaluate these stresses for any particular load, it is necessary to find $Q(a)$ from Eq. (13). For the trapezoidal loading of Figure 8, one finds from Eq. (13)

$$(16)\quad Q(a) = \frac{-2q}{\pi a b a^4}\,[\cos a(a+b) - \cos ab].$$

By allowing a to become zero, or b to vanish, one has

$$(17)\quad Q(a) = \frac{2q}{\pi a^3}\sin ab,$$

$$(18)\quad Q(a) = \frac{2q}{\pi a b a^4}\,[1-\cos aa].$$

Equation (18) represents an isosceles triangular loading and Equation (17) represents a rectangular loading, a special case being a point load P at the origin, namely,

$$Q(a) = P/\pi a^2.$$

Using Eqs. (16) and (15) a typical set of stress curves are shown in Figure 8. In Figure 9 one may make a comparison between the maximum shear stresses S_1 which occur on elements inclined at 45° with the vertical axis of symmetry (see Fig. 7) and the shearing stresses at the rock surface. These calculations were made for $\nu = 0.25$ and some small variation may be expected for other values of Poisson's ratio. With increasing depth h the shear stresses at the rigid base decrease as shown by the relative values when h = b and h = 5b. In the lower diagram of Figure 9, the depth of the layer is indicated by the vertical scale and two S_1 stress curves are shown for h = a and h = 5b, the latter being considerably distorted in comparison with

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wishes to acknowledge an error pointed out by the writer. There should be a negative sign (rather than positive) in the term $\sinh^2 a$ in the definition of M at the end of page 38. Subsequent formula should be altered accordingly with a rearrangement of curves C and D in Figure 2 of his paper.

[11] By translating the x axis to the top of the surface with $y' = h - y$, then

$$\underset{h\to\infty}{\text{Limit}}\, p_x = \int_0^\infty Q(a)e^{-ay'}(ay' - 1)a^2\cos ax\,da.$$

A similar result holds for p_y. These are Boussinesq results for symmetrical loads.

the former. If the vertical scale were enlarged to the proper ratio, the curve representing the case h=5b would resemble the S_1 curve of Figure 2, except that the initial values at the top surface

SURFACE DEFLECTION FOR A FINITE LAYER

Employing the last of Eqs. (11) one obtains a general displacement function. At the top surface this becomes

$$(19) \qquad Ev\Big|_{y=h} = -2(1-\nu^2)\int_0^\infty \frac{Q(a)}{\Delta}\,[(3-4\nu)\cosh ah \sinh ah - ah]\,a\cos ax\,da.$$

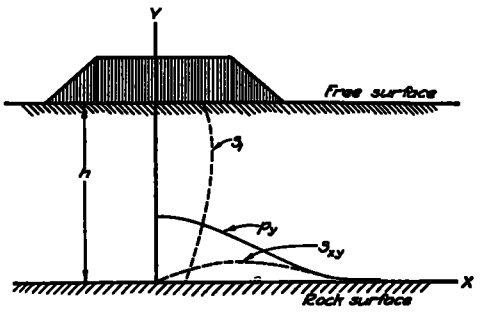


Figure 8

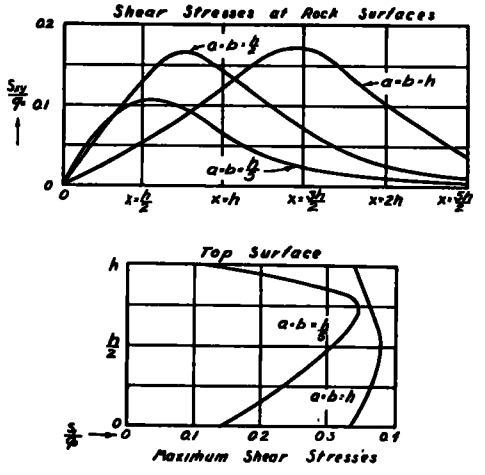


Figure 9

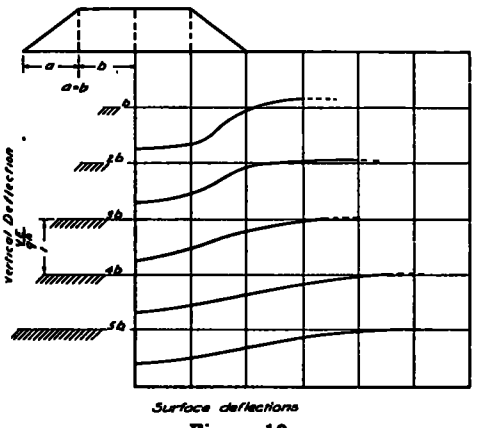


Figure 10

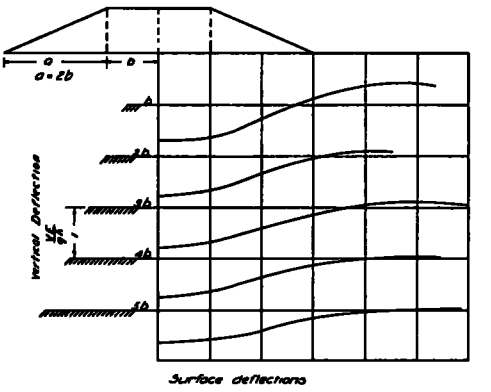


Figure 11

are not zero as in the infinite layer. This means that p_y and p_x are not equal directly under the load when the layer is finite, their difference increasing with decreasing depth of elastic layer. The maximum value of S_1 is much greater than S_{xy} at the base and one may expect shearing failure within the layer before similar failure occurs at the rock surface.

For a symmetrical trapezoidal loading $Q(a)$ is given by Eq. (16). For $\nu=0.25$ a series of the surface deflection curves are shown in Figures 10 and 11 for two different types of loads and for five increasing depths of layer. At the left of the vertical axis of symmetry the depth of the layer is shown and the scale indicates that for each layer the deflection is

given in terms of the height of that layer. For example in Figure 11 when h=3b the maximum value is vE/q=0.73h, and when h=4b, vE/q=0.65h. With increasing depth of layer the deflections continue to increase and the point of zero surface deflection recedes farther from the loading. This confirms the results of

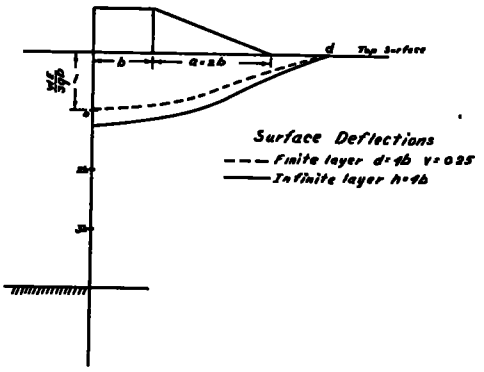


Figure 12

the infinite layer in which as the value d increased the deflections increased. Whereas the value of d was indeterminate, and only local relative deflections could be found in the infinite layer, the complete deflection surface for the finite layer can be determined. These latter curves do not oscillate with alternate positive and negative deflections. The extreme right portions of the curves of Figures 10 and 11 decay asymptotically along the horizontal. A comparison of the curve d=4b from Figure 11 and the lowest curve of Figure 5a is made in Figure 12.

SUMMARY

In this paper the stresses and surface deflections of a semi-infinite elastic medium and of an elastic layer of finite depth are obtained by the principle of superposition. In the first part of the paper the author determines the stresses induced in a supporting medium for surface loads which simulate triangular, rectangular, symmetrical and unsymmetrical trapezoidal surface loads. In particular it is shown that the depth at which the maximum principal shear occurs is at a depth approximately equal to one-half of the length of an equivalent rectangular load of equal intensity. This maximum shear value does not exceed 0.318q where q is the maximum intensity of the surface loading.

Only the relative surface deflection due to these loads can be determined. Two undetermined parameters remain to be found by experiment. One of these is $E'=E/(1-\nu^2)$ where E is the elastic compression modulus and ν is Poisson's ratio. Since this parameter appears only as a scale factor in the deflection, it may be designated as a foundation modulus. The second parameter capable of experimental determination is a distance d which locates a surface point having no displacement and relative to which all other surface deflections are referred.

In the second part of the paper, the stresses and surface deflection are determined for an elastic layer of finite depth supported on a rough rigid base. It is shown that the stresses are no longer independent of Poisson's ratio ν, and that shearing failure occurs within the layer before it occurs at the supporting base. Using $\nu=0.25$ the surface deflections are evaluated for several types of surface loading and varying depths of the elastic layer. All the results of the first part of the paper may be derived as a limiting case by allowing the depth h of the layer to become infinite.

APPENDIX *

The stresses p_y, p_x, and S_{xy} for the loading of Figure 3d can be obtained by summation of elemental loads employing Eqs. (1). The only change in the present case is that the angle θ is measured from

* The writer wishes to acknowledge the assistance of Mr. R. H. Tripp, graduate student at Iowa State College, in obtaining the equations in this appendix.

the direction of the loading which is no longer normal to the inclined surface plane. The results of Figure 3d may be obtained by superposing the results for triangular and parallelogram loads. The loading OAC of Figure 13 is equivalent to loads OAB+ABCD−ACD. This is permissible since it is assumed that the vertical intensity of the loading on the inclined surface is given by the configuration OACB.

zontal surface plane when k=0, that is,

$$p_y=\frac{-q_0}{2b\pi}(x_1a_1-2b\sin\theta_2\cos\theta_2),$$

$$p_x=p_y+\frac{q_0h}{b\pi}\log R_1/R_2, \tag{21}$$

$$S_{xy}=\frac{q_0}{2b\pi}(a_1h-2b\cos^2\theta_2).$$

With the notation of Figure 14 the stresses due to the parallelogram loading ABCD are

$$p_y=\frac{q_0}{\pi(1+k^2)^2}\Big[k(1+k^2)(\sin^2\theta_2-\sin^2\theta_1)+\frac{(1+k^2)}{2}(\sin2\theta_2-\sin2\theta_1)$$
$$-a_1(1+3k^2)-2k^2\log R_1/R_2\Big],$$

$$p_x=\frac{2q_0}{\pi(1+k^2)}\left[k\log\frac{R_2}{R_1}-a_1\right]-p_y, \tag{22}$$

$$S_{xy}=\frac{q_0}{\pi(1+k^2)^2}\Big[(1+k^2)(\sin^2\theta_2-\sin^2\theta_1)-\frac{k(1+k^2)}{2}(\sin2\theta_2-\sin2\theta_1)$$
$$-a_1k(1-k^2)-2k^2\log R_1/R_2\Big].$$

With the notation of Figure 13 the cases of a triangular load OAB on an inclined surface is given in the following equations.

Triangle OAB:

$$p_y=\frac{q_0}{b\pi(1+k^2)^3}\Big\{b(1+k^2)\Big[\frac{1+k^2}{2}\sin2\theta_2-k(1+\cos^2\theta_2)+k^2\sin^2\theta_2\Big]$$
$$-\frac{a_1}{2}\Big[(1+6k^2-3k^4)x_1+2h(3k^2-k)\Big]$$
$$+2bk^2(1+k^2)-k^2[4kx_1-h(3-k^2)]\log R_1/R_2\Big\},$$

$$p_x=\frac{q_0}{b\pi(1+k^2)^2}\Big\{[(1-k^2)h-2kx_1]\log R_1/R_2+2bk(1+k^2)$$
$$-a_1[(1-k^2)x_1+2kh]\Big\}-p_y, \tag{20}$$

$$S_{xy}=\frac{q_0}{b\pi(1+k^2)^3}\Big\{b(1+k^2)\Big[(1+k^2)\sin^2\theta_2-\frac{k(1+k^2)}{2}\sin2\theta_2-(1-k^2)\Big]$$
$$-\frac{a_1}{2}[2kx_1(1-3k^2)-h(1-6k^2+k^4)]$$
$$+2bk^2(1+k^2)-[k^2x_1(3-k^2)-2kh(1-k^2)]\log R_1/R_2\Big\},$$

where k=tanα (not to be confused with k of Eqs. (3)).

These formulas provide the results for a rectangular load on a horizontal surface plane when k=0.

To obtain the stresses p_t, p_n, S_{tn} (indicated in Figure 14) which are parallel and normal to the inclined surface plane one employs the equations

$$p_t=p_x\cos^2\alpha+p_y\sin^2\alpha+2S_{xy}\sin\alpha\cos\alpha,$$
$$p_n=p_x\sin^2\alpha+p_y\cos^2\alpha-2S_{xy}\sin\alpha\cos\alpha, \tag{23}$$
$$S_{tn}=(p_y-p_x)\sin\alpha\cos\alpha+S_{xy}(\cos^2\alpha-\sin^2\alpha).$$

Equations (20) reduce to known results [2] for a right triangle load on a hori-

With Eqs. (23) one may show that both the normal stress p_n and the tan-

gential stress S_{tn} vanish at the surface when external to the load and yield the correct values under the load. These values are

For parallelogram:

$$p_n = -q_0 \cos\alpha, \quad S_{tn} = -q_0 \sin\alpha \cos\alpha,$$

For triangle:

$$p_n = -\frac{q_0 x_1}{2b} \cos^2 \alpha, \quad S_{tn} = -\frac{q_0 x_1}{2b} \sin\alpha \cos\alpha.$$

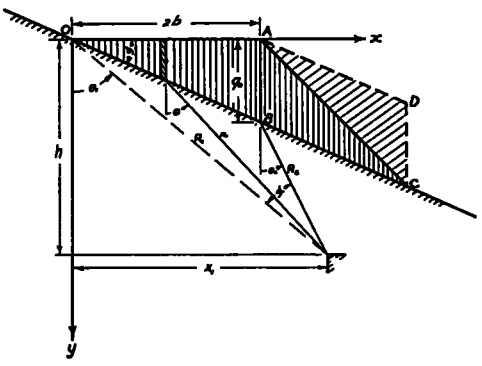


Figure 13

The tangential stress p_t along the surface plane changes from a compressive stress to a tensile stress at some point under the loading. In the case of the parallelogram this point depends only on the slope of the surface plane, and in the triangle loading it also depends upon the length of loading 2b.

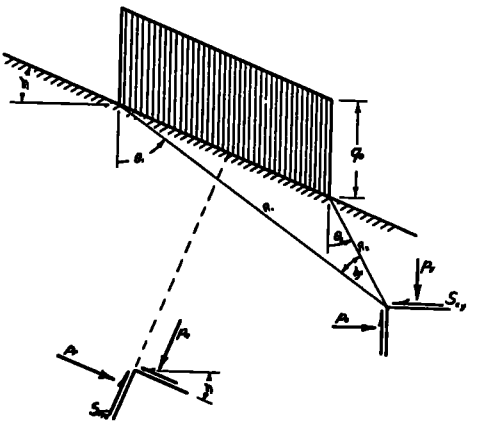


Figure 14

DISCUSSION ON SHEARING STRESSES AND SURFACE DEFLECTIONS DUE TO TRAPEZOIDAL LOAD

MR. L. A. PALMER, *Public Roads Administration:* The author has followed a general procedure which is as old as soil mechanics and which "neglects" the fact that stresses within the embankment and supporting soil may be more or less continuous. This apparent "neglect" is necessary if any sort of mathematical expressions for stress are to be obtained. One must consider the load due to a fill or embankment simply as a load and not as a projection of the undersoil. In a natural earth embankment this procedure may not be warranted.

It may be true that the stress trajectories within a finite earth mass, such as a fill, and those in the "infinite" soil mass, the supporting earth, are more or less continuous but to assume anything with respect to such continuity is almost useless. Therefore, this writer is in thorough agreement with the author's procedure. It would be well for those who object to this procedure to suggest a better one in some detail.

For soils that consolidate due to slow egress of water under a foundation load, it has apparently been customary in soil mechanics to take μ, Poisson's ratio, as $\frac{1}{2}$. No defense of this procedure is at present offered. Let S denote the total subsidence due to all causes, S_L the settlement due to distortion at constant volume with $\mu = \frac{1}{2}$, and S_C the settlement due to consolidation with $\mu = 0$. Then

$$S = S_L + S_C.$$

With reference to Figure 4 of the

author, the vertical displacement, w, at any point on the axis of symmetry may be shown to be

$$w = \frac{yp}{\pi Ea}\left[\frac{y^2}{2}\log_e\frac{\sqrt{y^2+(a+b)^2}}{\sqrt{y^2+b^2}} + \frac{(a+b)^2}{2}\log_e\frac{\sqrt{y^2+(a+b)^2}}{a+b} - \frac{b^2}{2}\log_e\frac{\sqrt{y^2+b^2}}{b}\right] + \text{constant.}$$

The value of the constant is obtained by giving a value to y of magnitude such that $w=0$ for practical purposes. For $y=0$, $w=Wmax=S_L=$the constant.

A more general expression for w for any point in the supporting earth and for any value of μ but which contains a constant instead of the author's "d" is easily derivable.

DR. KARL TERZAGHI: In the first part of the paper, dealing with the case illustrated by Figure 3d, the general reader would undoubtedly appreciate the inclusion of the Boussinesq equations for the stresses due to a force acting in the direction of the plane surface of a semi-infinite elastic medium, because these equations are not as well known as the equations pertaining to load perpendicular to the surface.

The last part of the paper (elastic layer on a rigid rock base) deals with a very difficult problem. The following solutions may be mentioned.

E. Melan, "Die Verteilung des Druckes durch eine elastische Schichte," *Beton u. Eisen,* 1919. Point and line load, no bond between layer and base.

K. Marguerre, "Druckverteilung durch eine elastische Schichte auf starrer, rauher Unterlage," Ingenieur Archiv, 1931. Line load, perfect bond between layer and base.

W. Passer, "Druckverteilung durch eine elastische Schichte." Sitzber. Ak. Wiss., Wien, Abt. IIa, Vol. 144, 1935. Point load, perfect bond between layer and base.

M. A. Biot, "Effect of Certain Discontinuities on the Pressure Distribution in a Loaded Soil," *Physics,* Dec. 1935. Point and line load, no bond and perfect bond.

None of these very competent investigators attempted to accomplish more than a computation of the distribution of the stresses over the rigid bottom. Yet their investigations are rather involved. The results contained in the paper by Mr. Holl go far beyond this modest accomplishment. Therefore the reader is naturally interested in the details of the mathematical investigations which led to the results.

The value of the paper could also be increased by references such as those mentioned above.

DR. HOLL: As stated in the paper, one has the alternative of referring the relative deflections to a surface point or to an internal point having no deflection. Mr. Palmer has given such an expression referring the deflection to an internal point on the axis of symmetry and for the special value μ (or ν) $=0.50$. The author's choice of referring the relative deflections to a surface point d was based upon the fact that surface deflections admitted experimental verification.

The references and mathematical derivations alluded to by Dr. Terzaghi were omitted from the abbreviated preprint. The author acknowledged these omissions at the presentation of the paper and explained the relationship of the plane deformation analysis of this paper to the problems of plane stress and axially symmetric stress situations of the quoted references.

SHEARING STRESSES AT THE BASE OF AN EMBANKMENT

By D. P. Krynine

Research Associate in Soil Mechanics, Yale University

SYNOPSIS

In most of the existing methods of designing foundations for embankments it is assumed that the embankment exerts only vertical pressures on its base. Particularly, this is done in applying the theory of elasticity to the design of embankments. As a result of such an assumption, shearing stresses at the base of the embankment should vanish, but in reality they exist. The purpose of this paper is to furnish a quick method of estimating these shearing stresses, and it is believed, that the solution thus obtained is satisfactory for all practical purposes.

A highway embankment or an earth dam tends to slide sidewise. The chief reason for it is lateral pressure which exists in every earth mass. In German literature there are some rather complicated methods of computing the shearing stresses at the base of an embankment.[1] So far as the writer is aware, this problem has not yet been theoretically approached by engineers in English speaking countries. In this paper a simple solution is proposed. It is approximate and its accuracy may be questioned from the point of view of exact elastic theories. It furnishes, however, a fairly clear view of the situation and allows an estimate of the maximum shear at the base of an embankment.

TRIANGULAR CROSS SECTION

To simplify the reasoning consider first an embankment of triangular cross section (Fig. 1(a)). Let its height be H ft., and designate by α the angle made by the slope with the horizontal. Suppose that the slope in question is characterized by the ratio: one height for n bases. Then:

$$\tan \alpha = 1:n \ldots\ldots\ldots (1)$$

Designate furthermore with γ the unit weight of the material so that 1 cu. ft. weighs γ lb. Imagine that the embankment is cut into two halves along its center line and consider conditions of equilibrium of one half only, for instance, of the left (Fig. 1(b)). Because of the symmetry, the left half pushes the right one horizontally, and vice versa; and there cannot be any vertical component at the center line. Hence the right part can be removed and replaced by the horizontal thrust, F_o, which it exerts on the left half. This thrust can be computed using the Coulomb formula for the horizontal backfill. This is accurate enough for all practical purposes; and it should be noticed that the Rankine formula furnishes the same result in this case:

$$F_o = \frac{\gamma \cdot H^2}{2} \cdot \tan^2\left(45° - \frac{\phi}{2}\right) \ldots\ldots (2)$$

As known, the symbol ϕ in Formula (2) means the average angle of friction of the given earth material. Place for the sake of brevity:

$$\tan^2\left(45° - \frac{\phi}{2}\right) = K_a \ldots\ldots (3)$$

so that Formula (2) will be:

$$F_o = \frac{\gamma \cdot H^2}{2} \cdot K_a \ldots\ldots\ldots (4)$$

This thrust is resisted by the adherence of the embankment to its base; so that the average shearing stress, τ, at the base will be obtained by dividing the value of the thrust, F_o, by the half width of the embankment which equals $H \cdot n$. Hence, the value of the average shearing stress,

[1] For instance, Leo Rendulic, Der Erddruck im Strassenbau und Brückenbau. Berlin, 1938.

τ, at the base of a triangular embankment is:

$$\tau_{av} = \frac{F_o}{H \cdot n} = \frac{K_a}{2n} \cdot \gamma \cdot H \ldots\ldots (5)$$

Each vertical section of the embankment such as AB (Fig. 1(b)) may be visualized as a retaining wall *dx* thick. Thrust, F, acting on this retaining wall from the center line toward the toe is greater than the opposite thrust F-dF. Suppose that the thrust F (or F-dF) makes an angle ϕ, that of internal friction of the earth material, with the normal to the retaining wall, AB. The horizontal force, $dF \cdot \cos \phi$, which pushes the retaining wall, AB, toward the toe of the embankment, is balanced by the shearing force $\tau \cdot dx$ at the base of the embankment:

$$dF \cdot \cos \phi = \tau \cdot dx \ldots\ldots (6)$$

from which:

$$\tau = \cos \phi \cdot \frac{dF}{dx} \ldots\ldots (7)$$

It is known that the thrust, F, is proportional to the square of the height of the retaining wall, AB, and the latter equals $\frac{x}{n}$ (Fig. 1(b)). Were the slope, MN, infinitely long, thrust, F, at all vertical sections such as AB, would be proportional to x^2 or in other words, according to Equation (7) the shearing stress, τ, at the base of the embankment would be proportional to x, because the derivative $\frac{dF}{dx}$ is proportional to x. In reality, slope MN is finite, and starting at a certain point, P, the shearing stress, τ, decreases towards the center line of the triangular embankment where it becomes zero due to symmetry. To locate point P, it is necessary to know the angle QPN, N being the top of the embankment. The slope of the line PN may be determined analytically or graphically by using one of the numerous methods to locate the Coulomb wedge behind a retaining wall with sloping backfill. The simplest solution would be to trace through N a line making an angle $45° - \frac{\phi}{2}$ with the vertical as in the case of a horizontal backfill. Then area QPN would correspond to the possible maximum wedge behind hypothetical walls imagined within the

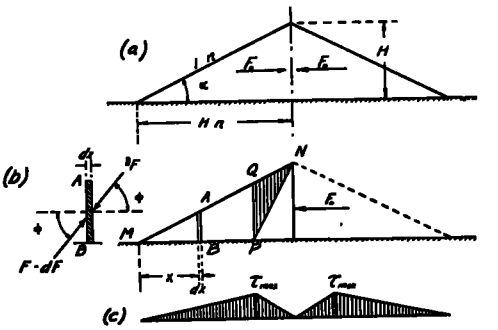


Figure 1. Triangular Cross-Section

embankment and it may be assumed that the difference in thrusts, F and F-dF, is also maximum at the section QP. The decrease of the shearing stress, τ, from point O toward the center line of the embankment does not follow a straight line law any longer. Nevertheless, for the sake of simplicity, the curve expressing the value of the shearing stress, τ, between point P, where this stress reaches a maximum, and the center line of the embankment, will be expressed by a straight line. Thus the diagram of the shearing stress, τ, for each half of the embankment will be a triangle (Fig. 1(c)). It is obvious that according to this visualization the maximum value of the shearing stress, τ_{max}, equals twice the average value, τ_{av}:

$$\tau_{max} = 2\tau_{av} = \frac{Ka}{n} \cdot \gamma \cdot H \ldots\ldots (8)$$

Recapitulating: In the case of a triangular embankment the shearing stress is at a maximum at a point of the base which may be approximately determined by tracing through the top point of the embankment a line making an angle of $45° - \frac{\phi}{2}$ with the vertical. The maximum

shearing stress, τ_{max}, may be expressed in terms of $\gamma \cdot H$, where γ is the unit weight of the earth material expressed, for instance in lb. per cu. ft.; and H the height of the embankment (in ft.). It equals $\frac{K_a}{n} \cdot \gamma \cdot H$ where K_a is determined by Formula (3), and n is the coefficient of the slope. Thus for $\phi = 30°$ and for a slope one height to two bases (n=2);

$K_a = 0.33$ and $\tau_{max} = \frac{1}{2} \times 0.33 \cdot \gamma \cdot H$
$= 0.165\gamma \cdot H$

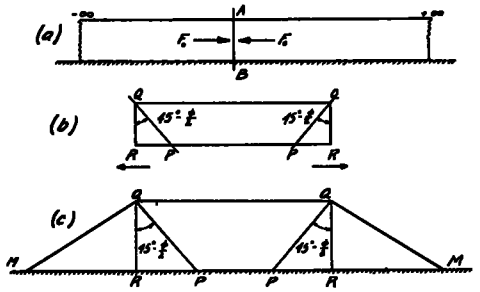


Figure 2. Trapezoidal Cross-Section

TRAPEZOIDAL CROSS-SECTION

Consider first an infinitely long fill of uniform height (Fig. 2(a)). At any vertical section, AB, there are two opposite and mutually equal horizontal thrusts, F_o. There is no shear at all at the base of such an embankment.

If, however, the embankment is finite though very wide, there is shear close to its edges. By tracing a line, QP, through the outside upper point, Q, of the embankment, a point, P, may be found where the shear stress, τ, starts to increase from zero to a certain value at the edge, R (Fig. 2(b)). The maximum value of the shearing stress in this case is at the edge. If a slope is added (Fig. 2(c)), there would be a quite small shearing stress at point, R, because the pressures from both sides of the section, QR, are almost equal. Hence in this case distance, PR, takes up but a very small part of the total pressure pushing the embankment toward its toes; and it seems advisable to neglect its effect at all. Thus the problem is reduced to the study of stability of the triangular prism, MQR, i.e. to the preceding problem.

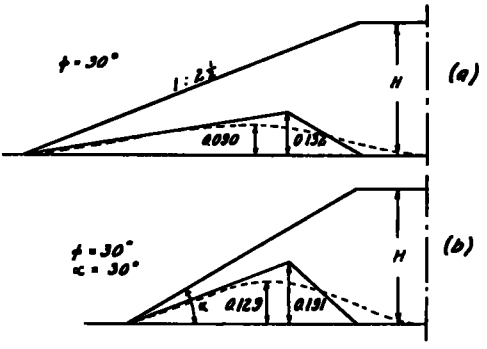


Figure 3. Shearing Stresses at the Base, in terms of $\delta \cdot H$. (Broken Lines are Rendulic's Data).

The situation differs very little if the crown is narrow. To be on the safe side, it is better to neglect the influence of the central part and consider the stability of the slope only, as in the case of a wide crown.

COMPARISON OF DATA

In Fig. 3(a) and (b) diagrams of shearing stresses as given by Rendulic and as computed by the proposed method are shown. The value of the maximum shearing stress, τ_{max}, as computed by Formula (8) is about 50 percent in excess of that given by Rendulic. One must take into consideration on one hand the uncertainty in the actual distribution of those shears, and the difficulty in determining in the laboratory the shearing resistance at the boundary of the fill and the earth mass, and on the other hand the simplicity of the method proposed. Under the circumstances, it seems advisable to give preference to the simpler method of computing stresses which at the same time furnishes a certain margin of safety.

SHEAR STRESSES IN PRACTICE

The existence of shearing stresses at the base of an earth dam has been proved by Dr. Terzaghi by observing movement of a pipe line under a dam in Africa as reported by him in one of his public conferences.

Shearing stresses at the base of an embankment may be dangerously increased by the overrolling during construction. The overrolled material tends to expand, and this causes the lateral sliding as happened in the case of the Tappan Dam.[2] This type of failure probably unknown to most engineers, merits attention.

The bond between a fresh fill and the existing earth mass increases as the time goes. Hence the maximum danger of sliding apparently corresponds to the construction period; therefore it appears proper not to consider the influence of the live load (traffic) on the shear at the base as done in this paper.

DISCUSSION ON SHEARING STRESSES AT THE BASE OF AN EMBANKMENT

MR. L. A. PALMER, *Public Roads Administration:* According to equation (2) of this paper, the lateral earth pressure is the vertical one multiplied by $\tan^2 (45° - \phi/2)$. This could be approximately true if the material comprising the fill is cohesionless, if the supporting soil is unyielding and if the movement of fill material in the horizontal direction is of a magnitude characteristic of a condition of failure. Actually, the fill material is seldom cohesionless, the supporting earth yields in the vertical direction under the fill load and the lateral movement of fill material at mid-section may vary from zero to almost any value.

It may be expected that settlement near the center line of the undersoil is of considerable magnitude and that as a consequence the material at the longitudinal mid-section of the fill will tend to drop vertically downward. Thus there is a condition comparable to the yielding of a trap door at the base of bin of sand and arching at the mid-section of the fill results. Due to this arching action, the thrust F_o (Figures 1a and 2a) is increased considerably. It is likely that under these conditions the lateral pressure at mid-section may exceed the vertical pressure. Marston[3] has shown that due to the large lateral thrust in the fill material above culverts the load on the culvert is in some instances materially decreased and that in other instances it is considerably increased, the increase or decrease of load on the culvert depending on the relative settlements of the material in the vertical section including the culvert and in the material adjacent to this section.

In equations (3), (4), (5) and (8) K_a is the active earth pressure ratio. If the supporting soil is unyielding the earth pressure ratio is K_a for the condition of failure but for small earth movements in the fill the earth pressure ratio would be intermediate in value between K_a and the coefficient of earth pressure at rest, assuming no displacement of the supporting earth.

Due to arching action when there is vertical displacement of the supporting

[2] Knappen, T. T., "Calculation of the Stability of Earth Dams." Second Congress of Large Dams, Washington, D. C., 1936, Vol. 4.

[3] See for example Bulletin No. 31, Engineering Experiment Station, Iowa State College, A. Marston and A. O. Anderson.

soil, the vertical pressure at the base of the longitudinal mid-section of the fill is decreased and the vertical pressure at the ground level and to right and left of the longitudinal mid-section is materially increased. This condition would be temporary however. The pressure intensity at the surface of supporting earth would tend to fluctuate as these conditions alternate in their occurrence.

THE DISTRIBUTION OF LATERAL EARTH PRESSURES

By M. G. Spangler

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SYNOPSIS

Experience in the construction of retaining walls often indicates that the quasi-hydrostatic pressure distribution predicted by the Rankine theory does not always obtain. This theory indicates that the center of the resultant pressure against a retaining wall would be applied at the top of the lower third of the wall. Various experiments have been made that show this center of pressure to be nearer the top of the wall in some cases. It is the author's belief that some of the discrepancies between the Rankine theory and actual experience may be clarified by the hypothesis advanced in this paper, which considers only the horizontal component of the earth pressure.

The resultant of the lateral pressure against a retaining wall may be considered to consist of two components, one due to the pressure of the backfill below the level of the top of the wall, and the other due to the loads superimposed above this level. The first of these components acts at approximately the lower ⅓ point of the wall, the pressure being distributed in accordance with Rankine's analysis for level backfill. The second component may act at any point above the base of the wall, depending upon the point of application of the superimposed load. The pressure distribution will be dependent on the type of the load. The resultant of these two loads may act at almost any point on the wall, but usually in the middle third. In general, the larger the superimposed load and the closer it is to the wall, the higher the resultant pressure will be on the wall.

The character of the lateral forces exerted by earth against any structure which restrains the earth from lateral movement has engaged the interest and attention of scientists and engineers for more than two centuries and the literature is replete with hypotheses, theories, and accounts of experiments and observations of the performance of retaining walls, abutments, and trench sheeting. In spite of this sustained and widespread interest, a basic discrepancy between the classical earth pressure theories of Coulomb and Rankine and the observed performance of some actual structures continues to exist without adequate or widely accepted explanation, while at the same time, many structures do behave as though acted upon by pressures distributed substantially in accordance with these classical theories. Until these contradictory facts are satisfactorily explained, new ideas will continue to be offered for discussion and comparison with experience, and it seems likely that the hope expressed by Mr. George Paaswell (1),[1] who, in his discussion of Mr. Jacob Feld's (2) retaining wall experiments, said ". . . . , makes one hope that it marks the end of the search for the proper method of determining the thrust on a retaining wall," may not be realized for some time to come.

In presenting the hypothesis of this paper it is not the intent of the author to recommend that the classic earth pressure theories be discarded forthwith and a new untried theory substituted therefor. Rather he hopes to supplement these theories in such a manner that the principal differences between them and actuality may be reconciled. Until this or some other plausible supplementary hypothesis has been substantiated by both experiment and the observation of actual performance, certainly the classical theories should not be discarded. They have served engineers too long and too well to be lightly overthrown. However, their age and past usefulness do not

[1] Numbers in parenthesis refer to list of references at end.

exempt them from critical examination and improvement, if that is possible.

The principal discrepancy between Rankine's theory and observed performance seems to be in the distribution of lateral earth pressure which is reflected in the position of the line of action of the resultant pressure against the lateral supporting structure. Mr. J. C. Meem (3), writing as a result of extensive experience in subway and retaining wall construction, concluded that the resultant of the lateral earth pressure to which the sheeting of an open trench or a retaining wall was subjected, acted at an elevation much nearer the top of the trench or wall than is indicated by theory, and, therefore, that the pressure near the top of the structure was much greater than is indicated by the quasi-hydrostatic pressure distribution of the classical theories. On the other hand, Mr. E. P. Goodrich (4) in discussing Mr. Meem's paper, stated: "In a few instances, in the speaker's experience, he has encountered earth pressures which were evidently greater near the ground surface than at lower levels. On the other hand, the opposite condition has been observed in a considerably greater number of cases, and numerous experiments of a very careful nature have shown the close agreement between fact and theory in this regard." The varying experience of Messrs. Meem and Goodrich seems to be indicative of the state of our knowledge of the distribution of lateral earth pressures at the present time.

The experiments conducted by Mr. Jacob Feld at the University of Cincinnati in 1921 and 1922 revealed a similar contradictory situation. Feld's measurements of the position of the resultant of the horizontal pressure on vertical walls loaded by backfills having horizontal surfaces showed the resultant to be at or very near the theoretical height indicated by Rankine's theory and formula. However, in one significant test (test No. 5), wherein additional fill material in the form of a sloping surcharge was placed above the level of the top of the wall, the position of the resultant pressure was raised to a point considerably higher than the lower one third point. In this experiment the backfill was constructed to a height of 5 ft. on the experimental wall with the top surface of the fill sloping downward and backward at an angle of 33 deg. 40 min. Additional material was then added to the backfill and the slope of the surface was varied by about three-degree increments up through the horizontal and on upward until a slope of 34 deg. above the horizontal was attained. The height of fill at the wall remained constant throughout this process. The position of the resultant lateral pressure on the wall was observed to be at the lower one third point for all backfill surface slopes below and including the horizontal. As the upward slope increased, the position of the resultant gradually rose, reaching a height above the base of the wall equal to 0.43 H when the slope was 34 deg. above the horizontal. The data in Table 1 are taken directly from Table 8 on page 1475 of Feld's article. These experiments were very carefully performed, using a "damp" to "humid," nearly cohesionless sand for backfill material.

In 1925, Mr. J. V. McNary (5) published the results of some measurements of lateral pressures on the back of two bridge abutments with horizontal backfills, in which the pressures were measured by means of Goldbeck pressure cells. These measurements revealed a distribution of lateral pressures substantially in accord with that indicated by the Rankine theory.

Mr. Terzaghi (6) has attributed the apparent contradictions between theory and practice to variations in the amount and character of the horizontal yield or movement of a retaining wall or other type of lateral support. His theory of

indeterminacy is somewhat difficult to comprehend and apply. It is recognized that every wall will yield to some extent, but it is impossible to predict with any reasonable degree of certainty what the magnitude of lateral yield of a retaining wall will be, within the degree of precision which Terzaghi indicates is necessary. So much depends upon the character of the soil upon which the base of a wall rests.

TABLE 1

DATA SHOWING OBSERVED POSITION OF LATERAL PRESSURE RESULTANT IN TEST NO. 5 OF FELD'S CINCINNATI EXPERIMENTS

Slope of backfill surface	Position of resultant. Ratio of height above base to total height of wall
− 33° 40′	0.33
− 31°	0.33
− 28° 5′	0.33
− 25°	0.33
− 21° 50′	0.33
− 18° 30′	0.33
− 14° 55′	0.33
− 11° 20′	0.33
− 7° 40′	0.33
− 3° 50′	0.34
0°	0.34
+ 3° 50′	0.35
+ 7° 40′	0.35
+ 11° 20′	0.40
+ 14° 55′	0.40
+ 18° 30′	0.40
+ 21° 50′	0.42
+ 25°	0.40
+ 28° 5′	0.44
+ 31°	0.44
34°	0.43

If the author understands correctly the procedure followed in the experiments conducted by Terzaghi (7) and Darwin (8), which demonstrated the effect of the lateral yield of a wall upon the lateral pressure, the experimental walls were held rigid while the backfills were placed behind them. Then with the backfills completed, the walls were permitted to yield by loosening certain restraining devices. The order of occurrence of the yield and the time relationships between pressure and yield as permitted to develop in the experiments, therefore, were not analogous to the situation which prevails in actual retaining wall construction, wherein a substantial proportion of the total yield develops gradually as the backfill is placed. It is at least possible that the effect of yield on pressure on the wall may not have been the same in the experimental trials as in most actual situations. The pressure on a wall which yields gradually as a fill is constructed would probably be greater than the pressure on a wall which is permitted to yield after the fill is completed.

The foregoing discussion of a few selected references in the literature is given for the purpose of illustrating the situation in regard to the relationship between Rankine's theory and actual practice, and the results of experimental research, as it exists today. In brief this situation may be summarized by saying that there is considerable evidence to the effect that the Rankine distribution is realized in many actual construction and experimental cases. At the same time, there is evidence that the Rankine distribution is not realized in other cases, and when it is not, the line of action of the resultant pressure is always above the lower third point and the unit pressure near the top of the restraining structure is greater than indicated by theory.

The author (9) has, during recent years, conducted some experiments at the Iowa Engineering Experiment Station in which the distribution of lateral pressure upon a retaining wall was measured, when loads of various types were applied to the horizontal surface of the gravel backfill behind the wall.

These experiments brought into focus a new concept of the lateral pressure exerted upon a retaining wall by superimposed or surcharged loads applied near the wall, and it is possible that as this

concept is developed and grows more clear, some of the discrepancies between the Rankine theory and practical experience, may be clarified. The experiments indicated that the pressure on a retaining wall due to a concentrated load applied at the surface of a level backfill is distributed substantially in accordance with the empirical formula

$$h_c = \frac{KPx^2z}{x^nR^5} \quad \ldots\ldots\ldots\ldots 1$$

in which

h_c = normal unit pressure on the wall at any point.
P = applied concentrated load.
x = distance from load to back face of wall.
y = lateral distance from any point on the wall to the normal vertical plane containing the load.
z = vertical distance from any point on the wall to the horizontal plane containing the load.
$R = \sqrt{x^2+by^2+z^2}$
K, n, and b are empirical constants. K is a dimensional constant which may be written KC^n, where C is the number of units of length in 1 ft. The other constants n and b are abstract.

The results of these experiments are, in a general way, confirmed by the fact that the pressure distribution indicated by formula 1 is very similar to that indicated by the Boussinesq theory of distribution of a point load through an elastic solid, and by the fact that some laboratory experiments reported by Emil Gerber (10), show lateral pressure distribution similar in both character and magnitude to that measured for one of the load positions in the Ames experiments.

The values of the empirical constants, K and n, observed in these experiments were 1.1 (foot units) and $\frac{1}{2}$ respectively. The evidence concerning the value of b, which has to do with the rate at which pressure due to a concentrated surface load dissipates along the wall in a longitudinal direction, was somewhat limited in the experiments. Such evidence as is available indicates a value of b not less than 1.0 nor more than 2.0. None of these empirical values is conclusive, since only one type of backfill material was used in the study. Much more experimental work and study of retaining wall performance will be necessary before design values of the constants can be recommended. However, the principles upon which the hypothesis of this paper is based have been adequately demonstrated.

If it be assumed that a uniformly distributed parallel strip load acts the same as a series of equal, closely spaced, concentrated loads, formula 1 may be integrated in the y direction,

$$h_1 = KP\int_{-y_1}^{y_0} \frac{x^{(2-n)}z}{R^5}\,dy \quad \ldots\ldots\ldots 2$$

When y_1 and $y_0 = \infty$, equation 2 becomes

$$h_1 = \frac{1.33KP}{\sqrt{b}} \frac{x^{(2-n)}z}{R_1^4} \quad \ldots\ldots\ldots 3$$

in which

$$R_1 = \sqrt{x^2+z^2}$$

The measured pressure on the experimental wall, when a uniformly distributed strip load about eight inches wide and ten feet long was applied on the backfill surface and parallel to the wall, was found to be equal in both magnitude and distribution to the pressure calculated by formula 3, which indicates the validity of the assumption that a strip load acts the same as a series of concentrated loads.

With this experience as a background, we may speculate as to the pressure on a wall due to an area load, such as a building foundation, a railroad track, or simply additional or surcharged fill ma-

terial, by assuming that an area load acts the same as a series of parallel strip loads. Mathematically, this idea may be expressed by the integral of equation 3 in the x-direction

$$h_a = \frac{1.33KPz}{\sqrt{b}} \int \frac{x^{(2-n)}}{R_1^4}\, dx \quad \ldots\ldots 4$$

This expression has not been integrated, but it may be evaluated by making a summation of finite load strips about one foot wide in accordance with the formula

$$h_a = \frac{1.33KP}{\sqrt{b}} \sum \frac{x^{(2-n)}z}{R_1^4} \quad \ldots\ldots 5$$

A few experiments, very limited in scope, were conducted in 1938 in which the distributions of the horizontal pressures on a retaining wall were measured when area loads were applied to the horizontal surface of the gravel backfill behind the wall. The pressures were measured by means of stainless steel friction tapes in a manner similar to that reported in Bulletin No. 140 of the Iowa Engineering Experiment Station.

The loads were applied by constructing bins on the backfill surface at the level of the top of the wall and filling them with gravel, which was weighed as it was placed. Then the total weight of bins and gravel was assumed to be uniformly distributed over the area of the bins. These bins were 12 ft. long parallel to the wall and extended back from the wall as much as 20 ft.

The measured horizontal pressures on the wall for various area loads are shown in Figures 1 to 6 and are compared with the pressures calculated by equation 5. Although very limited in extent, these measurements reveal a distribution of horizontal pressure which is similar in character to that indicated by equation 5, and it seems likely that the assumption that an area load acts the same as a series of parallel strip loads will be substantiated when more research data become available.

In all of the experiments and the empirical equations resulting therefrom, and the Boussinesq theory as well, it is evident that the horizontal pressure distribution on a wall due to a surface load is not dependent upon the total height of the wall, but that it is dependent to a very great extent upon the position and magnitude of the super-imposed load.

On the basis of the results of these experiments and their extension into the field of area loads, and a study of the evidence provided by the accounts of

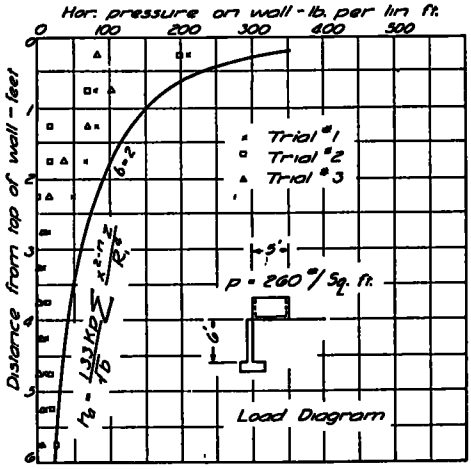


Figure 1

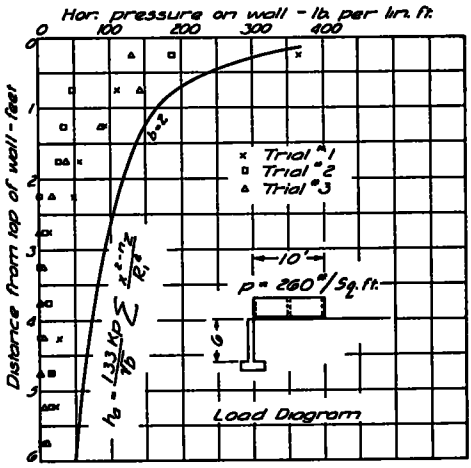


Figure 2

previous researches and observations of retaining wall performance as they appear in the literature, the following hypothesis which has to do only with the

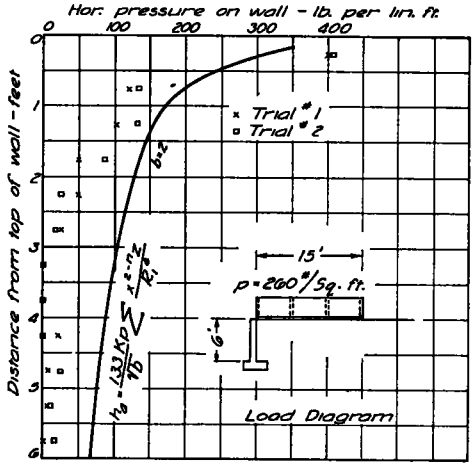


Figure 3

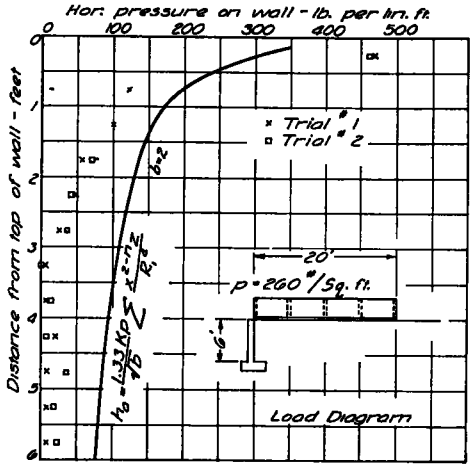


Figure 4

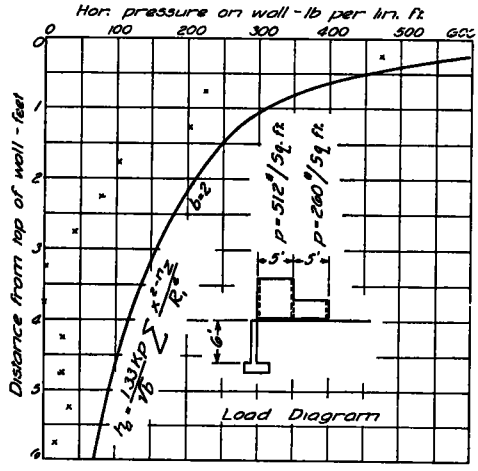


Figure 5

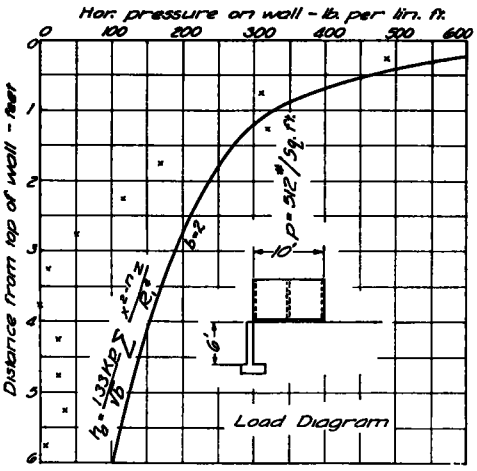


Figure 6

horizontal component of the earth pressures is proposed. The presence or absence of a vertical component of the earth pressure, whether due to friction between the earth and the wall or to some other cause is not under consideration.

The resultant of the lateral pressure on a retaining wall may be considered to be composed of two components; one due to the pressure of the backfill below the level of the top of the wall and the other due to loads superimposed above the level of the top of the wall and adjacent to it, including surcharge fill loads. The first of these components appears to act approximately at the top of the lower $\frac{1}{3}$ height of the wall, and the pressure is distributed in a quasi-hydrostatic manner in accordance with Rankine's analysis for a level backfill. The second

component may act at any distance above the base of the wall depending upon the position, both horizontally and vertically, of the super-imposed load relative to the wall, and the distribution of pressure will be as indicated by formula 1, 3 or 5 depending upon the type of the load. The resultant of these two components may act at almost any elevation, depending upon their relative magnitude and position, but will usually act somewhere within the middle $\frac{1}{3}$ height of the wall. In general, the greater the magnitude of the super-imposed load and the closer it is to the wall, the higher will be the position of the total resultant pressure and vice versa.

As an illustrative example of the application of this hypothesis, several purely imaginary cases are shown in Figure 7. The distribution of the total lateral pressure on a vertical wall 20 ft. high, when a super-imposed load such as a building foundation is applied at the surface of the backfill is shown for several positions of the foundation and for two magnitudes of foundation pressure. In each case the lateral pressure of the earth backfill is assumed to be distributed in accordance with Rankine's theory, with the resultant pressure acting at the top of the lower $\frac{1}{3}$ height of the wall. Then the lateral pressures due to

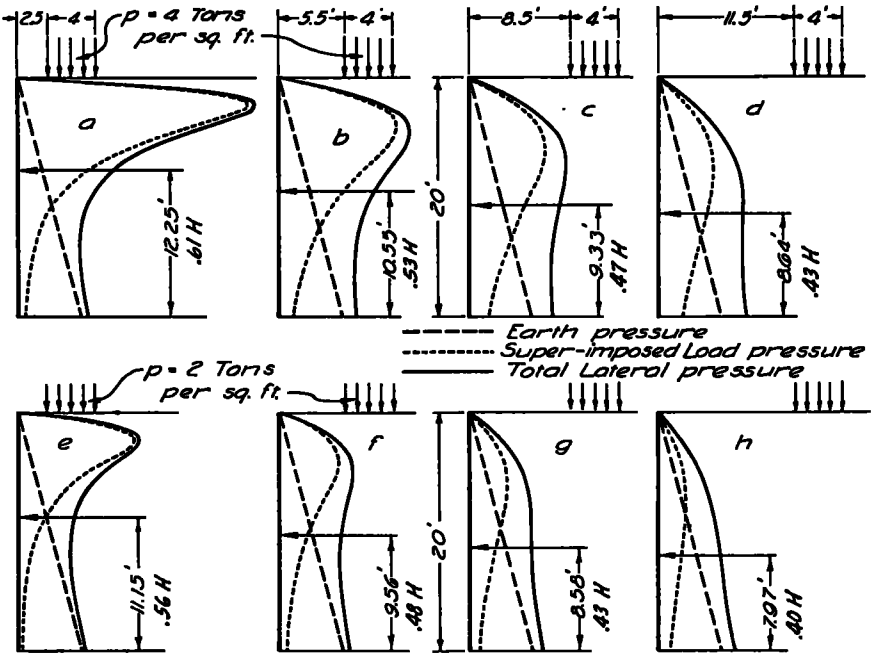


Figure 7

the super-imposed foundation load is added to the earth pressure, giving the total pressure distribution curves as shown. These total pressure curves vary widely in shape within the range of conditions assumed, and illustrate the important effect of the magnitude and position of super-imposed loads upon the distribution of lateral pressure and upon the position of the line of action of the resultant of the lateral pressure.

It is not possible to make a direct quantitative comparison between this hypothesis and the results of Mr. Meem's experience as reported in reference (3)

but it seems quite probable that the contradiction between his observations and Rankine's theory may have been due to the lateral pressure effect of superimposed loads, since most of his experience was in connection with subway construction in city streets with buildings occupying the adjacent property.

A comparison between the hypothesis of this paper and Mr. Feld's (2) experiment No. 5 is shown in Figure 8. When

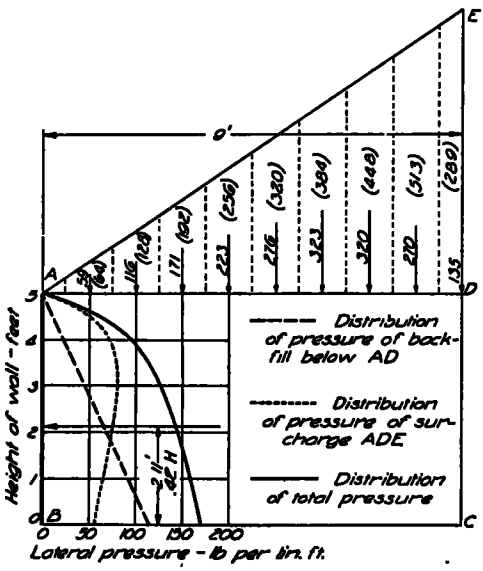


Figure 8

the backfill in his experiments was level, he noted that the position of the resultant lateral pressure was at a point 0.34H above the base of the experimental wall (see Table 1), indicating a distribution substantially in accordance with the Rankine theory, as shown by the dashed line in the figure. Then when the backfill was built upward on a slope of 34 deg. with the horizontal, the position of the resultant was observed to rise to a point 0.43H above the base.

If we consider the surcharged backfill material above the level AD as a superimposed load and calculate the lateral pressure on the wall due to this load by means of formula 5, the pressure distribution indicated by the dotted line in the figure is obtained. Then adding the pressures due to the material below the level AD to the pressures due to the surcharge, the distribution of the total pressure is obtained and is shown by the solid line. The resultant of this total pressure on the wall acts at a point 0.42H above the base, which is nearly coincident with the position observed by Feld.

In making these calculations for distribution of pressure due to the surcharged fill load of Feld's test No. 5, the weight of the fill above the top of the wall (shown in parenthesis in the figure) has been reduced by the probable frictional support at the sides and back of the bin in accordance with the principles of Janssen's analysis of pressures in grain bins and Marston's theory of loads on underground conduits. In all probability there were other friction losses along the sides of the bin below the level AD and back of the friction reducing section of the sidewalls provided in Feld's apparatus which cannot readily be calculated, and these losses may account for the fact that the total calculated lateral pressure on the wall due to the surcharge is about 345 lb. per lin. ft., whereas Feld measured only 214 lb. per lin. ft. However, it is not believed that the general shape of the total pressure distribution curve or the position of the resultant pressure would be greatly affected by these friction losses.

In presenting this hypothesis to the profession, the author hopes that it will stimulate discussion both from the theoretical and the practical standpoint. Particularly it would be interesting to hear from engineers who have observed pressure distribution which is at variance with classical theory. In such cases, have there been appreciable super-imposed or surcharged loads in the vicinity of the retaining walls or other types of restraining structures? Where possible, a quanti-

tative comparison of actual pressure distribution with that calculated by this hypothesis would be very interesting and valuable. Also, this hypothesis suggests a wide open field for both experimental and analytical research, which may lead to a more satisfactory reconciliation between theory and practice than exists at present.

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DISCUSSION ON DISTRIBUTION OF LATERAL EARTH PRESSURES

PROF. KARL TERZAGHI: In connection with the effect of surcharges on lateral pressure, I suggest that Mr. Spangler should include in his paper a comparison between his theory and the test results obtained by E. Gerber (Untersuchungen über die Druckverteilung im örtlich belasteten Sand. Zürich, 1929), because these results were obtained by means of remarkably sensitive measuring devices, and the scattering of the recorded values was far less important than in Mr. Spangler's test.

Mr. Spangler refers repeatedly to the contradictions between the classical earth pressure theories and reality. Contradictions of that type are invariably due to lack of clarity concerning the limits of validity of the theories involved. Since these limits have been known for several years, we can no longer speak of contradictions. A brief account of these limits is contained in my paper, "General Wedge Theory of Earth Pressure," *Proceedings* Am. Soc. C. E., October 1939. Figure 9 of this paper also contains the results of the measurement of the lateral pressure on the timbering of a cut and in response to Mr. Spangler's inquiry, it may be stated that there was no surcharge on either side of the cut. Otherwise this very essential detail would have been mentioned.

PROF. D. P. KRYNINE, *Yale University:* I wish to emphasize the following important features of this interesting paper.

First. Professor Spangler has concluded that a loaded area behind a retaining wall acts as a series of strip loads. This means that the principle of superposition holds in the case of sands and gravels, at least as far as horizontal

pressures are concerned. This conclusion is very important since sands and gravels are supposed not to obey Hooke's law and from the point of view of classical theories the principle of superposition cannot be applied to them.

Second. Experimental curves obtained by Professor Spangler are very similar to Boussinesq's pressure distribution. In all cases Spangler's ordinates are greater than those furnished by the theory of elasticity. This circumstance has been already discussed in engineering circles; but no definite conclusion has been reached. It should be very clearly borne in mind that Professor Spangler's empirical formula is valid only under given circumstances and cannot be generalized.

Third. Professor Spangler's basic hypothesis is quite correct but only in the case when the pressure on the retaining wall as caused by the backfill is hydrostatic. In other words, the retaining wall in question must be perfectly immovable or move only in a certain prescribed way. Professor Spangler's hypothesis does not hold in the case of flexible walls as he himself states.

Fourth. Professor Spangler's curves have maximum ordinates at the top of the wall. The classical theories require, however, that the horizontal pressure at the top of a backfill loaded at the surface with external loads be zero. There is no contradiction there however: Professor Spangler's experimental curves, if prolonged, would probably tend to the outside point at the top of the loading bins. This is an additional proof of the statement that in the case of an earth mass loaded also with earth, stresses must be computed considering the earth mass and the load (for instance, an embankment) as a whole.

MR. SPANGLER: We might go back a little in connection with Dr. Terzaghi's discussion and point out that comparison was made several years ago between our retaining wall experiments at Ames and the laboratory work of Emil Gerber at Zurich to which he refers. That comparison was shown at the meeting last year and has been published in the proceedings of the 18th annual meeting on page 58 of part II, and elsewhere.

I am very much interested in Professor Krynine's explanation of these pressure curves which bend so sharply to the right near the top of a retaining wall when the backfill is loaded with an area load contiguous to the structure and I am sure his suggestions will help to clarify some of our thinking on this subject.

FACTORS OF SAFETY TO BE APPLIED IN THE UTILIZATION OF SHEAR TEST DATA

By Donald M. Burmister

Assistant Professor of Civil Engineering, Columbia University

SYNOPSIS

The important problem is completely and accurately to determine the shearing characteristics of the soil. Careful consideration must be given to the methods of shear testing and to the analysis and interpretation of test results. Recent researches by leading authorities in the field of soil mechanics have given us certain fundamental principles. The ultimate shearing failure in soil is governed by the effective stresses carried entirely by the grain structure of the soil. A substantial part of the total applied stresses may be temporarily carried by hydrostatic stress in the pore water, which does not contribute in any way to mobilizing shearing strength. The proportion of the total applied stresses that is effective depends upon the relation between shearing stress and the volume-change that always accompany shearing deformations. The shearing characteristics are intimately related to consolidation characteristics, and are determined by the initial voids ratios and the load history by which this initial condition has been reached.

The shearing characteristics of certain cohesionless and certain cohesive plastic soils, both in the remolded and undisturbed states, are given in some detail, and are discussed in relation to shear failure phenomena. The results are analyzed as to values to be used in the analysis and solution of practical problems.

A few years ago it was thought that shearing phenomena were relatively simple and fairly well understood. But today we know that they include some of the most controversial and least understood questions of soil mechanics, and as a consequence many conflicting results have been obtained. However recent researches have established certain fundamental principles governing shearing failure.

If we are intelligently to apply a factor of safety, which is something more than a mere guess or a factor of ignorance and if we are to learn how to apply our knowledge correctly to the solution of practical problems, we must give careful consideration to:

1. The nature of shearing phenomena and conditions of failure, and the lessons learned from experience.
2. The methods used in obtaining information on the shearing characteristics of soils.
3. The analysis and interpretation of results of shearing tests.

An analysis of any given situation involves first of all attempting to understand the nature of the problem and trying to reduce the uncertainties to a minimum. Second, it involves making an estimate of the forces and physical factors governing the conditions of failure. And third, it involves attempting to define the limiting conditions in which shearing failure can possibly occur. In order to build up a body of authoritative knowledge on the subject, it is essential to make careful accurate observations in the field and systematically to collect data on the phenomena of shearing failures.

For cohesionless soils the researches of Casagrande (1) [1] (2) (3), Taylor (4), Watson (5), and for cohesive soils, the researches of Terzaghi (6) (7), Hvorslev (8), and Jurgenson (9) have given us fundamental principles governing shearing phenomena.

1. The ultimate shearing failure in soil is governed, not by the total

[1] Numbers in parentheses refer to list of references at end.

external applied stresses, but by the effective stresses carried entirely by the grain structure of the soil.

2. The relation between applied stresses and the volume-changes that accompany shearing deformations must be known in order to make an intelligent estimate of what proportion of the applied stresses may be effective for mobilizing shearing resistance.
3. The behavior characteristics of a soil in shearing depend upon the initial moisture content or voids ratio, the load history of the soil, and the consolidation characteristics of the soil.

We are concerned primarily with the safety of a mass of soil against overstress and failure by plastic flow or by rupture. This requires the determination of the shearing strength of soil under different possible conditions. The behavior characteristics of a mass of soil in shear depend upon the permeability and the compressibility of the material, the size of the mass involved, the voids ratio of the material with respect to a critical voids ratio, the volume-changes which take place during shearing, and whether the shearing failure tends to take place slowly or can take place rapidly. By varying shear test conditions indiscriminantly or by an arbitrary technique one can obtain almost any value of the angle of friction and of cohesion, between zero and its ultimate value. Such values can not be considered as soil constants and their use is dangerous. The most important tasks confronting us today are first, to develop scientifically correct methods of shear testing, which will yield significant values having a direct bearing on our problems, and second, to learn how to apply correctly the results in the solution of practical problems.

Terzaghi (6) and Casagrande (3) have stated certain conditions, which must be satisfied in order to obtain not more than one value for the shearing strength of a given material for each value of either the applied normal pressure or the minor principal stress:

1. All specimens must have the same initial moisture content or voids ratio.
2. This initial condition must be reached by the same loading procedure.
   (a) By increasing the consolidation pressure from zero to its ultimate value in a natural state of consolidation, and shearing the specimen under this pressure.
   (b) By pre-consolidating the specimen to a certain pressure and then unloading it to a certain smaller pressure under which the specimen is sheared—a state of over-consolidation.
   (c) By compacting the material to the same degree by a standard method, and then following methods (a) or (b).
3. The shear test must be run so slowly that total applied stresses are effective stresses, allowing sufficient time for the volume-change adjustments to take place and for full development of effective stresses. That is, excess hydrostatic stresses or neutral stresses in the pore water are kept to a very low value, either by slow continuous loading, or by very small increment shear loadings.

These investigators (3) (6) have emphasized the fact that the analysis of the shearing strength of either a test specimen or of a large mass of soil in nature must be based on a knowledge of effective stresses and on the possible changes of stress conditions with time, because

otherwise one can not be sure that applied stresses are fully effective. A substantial part of the applied stress may be temporarily carried by the water in the voids of the soil as a neutral stress, which does not contribute in any way to mobilizing shearing strength.

COHESIONLESS SOILS

In a laboratory test on cohesionless soil in a saturated condition a small mass of soil is involved and water can escape freely from the soil pores with volume decrease or flow into the soil with volume expansion at almost any rate of shearing. Externally applied stresses are therefore effective stresses. While the direct-shear test method is not entirely satisfactory and shows inconsistencies, the results for Ottawa sand in Figure 1 illustrate characteristic differences in the behavior of the material in the loose and dense states. In Figure 1a the ratio of the shearing stress to the normal pressure is plotted against the horizontal shearing deformation. Failure having the nature of plastic flow occurs in the loose state, where the stress gradually increases to a maximum value at a rather large horizontal deformation. Failure by rupture occurs in the dense state where there is a considerable drop in the shearing strength after the maximum has been reached, the maximum being considerably higher than that in the loose state. Taylor (4) has pointed out that while the maximum shearing strength is greatly influenced by the initial voids ratio, the more conservative ultimate is little affected, and approximately equals that of the loose condition.

The second characteristic difference is shown by the volume changes in Figure 1c, which accompany shearing deformations in the dense and loose states at the maximum stress condition. In the loose state the volume decreases and the mass of grains tends to assume a more dense state during shearing. In the dense state there is a slight initial volume decrease after which a marked expansion or loosening up of the dense structure occurs at the maximum stress.

The third characteristic difference is the effect of density on the angle of fric-

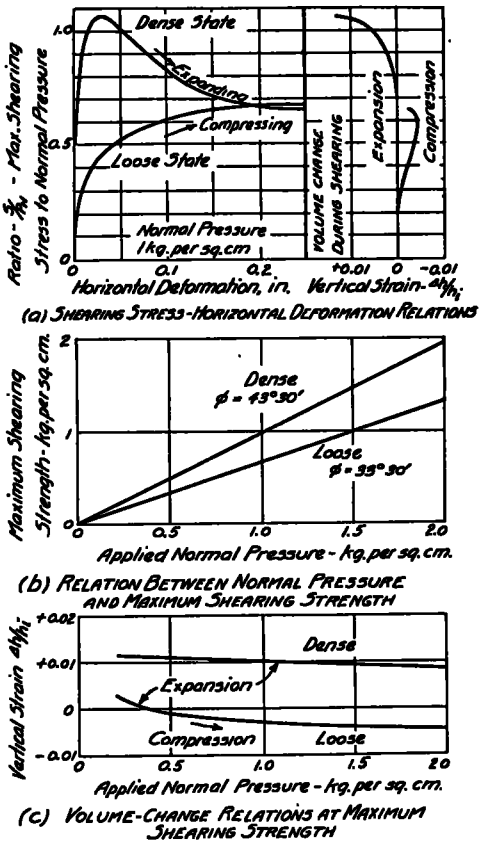


Figure 1. Shearing Characteristics of Standard Ottawa Sand, 20-30 Mesh

tion. Failure of masses of loose grains such as soils in shearing depends upon the internal frictional resistance to displacement of the grains that can be developed. In order to determine the angle of friction, it is necessary to establish the relation between the applied normal pressure and the maximum shearing strength of the material as shown in Figure 1b for Ottawa sand. This relation

may be expressed by Coulomb's simple equation for cohesionless soils—

$$S = p_N \tan \phi \quad \ldots\ldots\ldots\ldots 1$$

where

S is the maximum shearing strength,
p_N is the effective normal pressure,
ϕ is the angle of friction based on effective pressures.

The average values of the angle of friction for Ottawa sand in the loose and dense states obtained by direct-shear tests are as follows:

	Loose	Dense
Angle of friction—ϕ.....	33° 30′	43° 30′
Voids ratio—e	0.66	0.49

For materials which show a characteristic dropping off of shearing strength after the maximum is reached, design based on the maximum value may be decidedly on the unsafe side, while for designs based on the more conservative ultimate shearing strength, which approximately equals the loose maximum, the shearing strength can not drop below the ultimate, if progressive failure is initiated in some critical zone.

It is important in applying the results to practical problems to consider the volume-change relations at maximum stress as shown in Fig. 1c. In dealing with large masses of cohesionless soils in nature the time required for the volume-change adjustments to take place during shearing is very much longer than usual. Casagrande (1) (2) (3) has demonstrated that under such conditions a large mass of even sandy soil in a loose state may behave temporarily as relatively impermeable and hence may fail by a sudden flow failure as a result of sudden disturbance. A tendency toward volume decrease in the loose state during shearing would induce positive excess hydrostatic or neutral stresses in the pore water and only a part of the weight of the mass of soil would be effective in mobilizing shearing strength, in which case the shearing strength might be suddenly reduced to a very low value. On the other hand a tendency for volume expansion in the dense state during shearing would induce negative stresses, thereby making the effective stresses temporarily greater than the external stresses and giving the soil temporary additional strength during a disturbance.

The critical density has been defined by Casagrande (1) (2) (3) as the boundary region between the voids ratios in which volume decrease and volume expansion occur during shearing. It has been shown that the critical voids ratio is not a constant value, but decreases with increase in the applied stresses. The critical voids ratio is derived from the volume conditions at maximum stress, which, for the critical state is equal to the initial voids ratio.

There are many difficulties in accurately determining the critical density by means of direct-shear tests, because of the secondary effects, and because the stress conditions in the material during shearing are not definitely known. The most satisfactory method so far devised is the tri-axial compression test for cohesionless soils developed by Casagrande (2) (3). However by analyzing the results of the direct-shear tests on Ottawa sand an approximate relation is obtained in Figure 2a, similar to that developed by Casagrande (2) (3), giving the relation between the voids ratio at the start of the test, the volume-changes during the test, and, in this case, the applied normal stress, which is constant for each test. Each point on the line of zero volume-change represents a critical voids ratio, and the corresponding effective value, which the normal stress can assume during a flow failure, if such a condition could somehow develop within a large mass of saturated cohesionless soil as a result of a sudden disturbance.

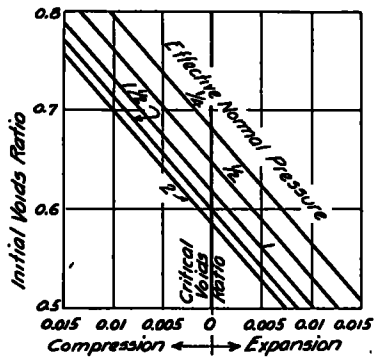
A curve representing the relation between the critical voids ratio and maximum effective shearing strength for the flow failure condition for Ottawa sand is obtained from Fig. 2a by substituting shearing strength from the relation— $S = p_N \tan \phi$ [or $S = (1 + \sin \phi) \tan \phi$, where the minor principal stress is used] as plotted in Fig. 2b. The curve is conveniently graduated for relative density in terms of the limiting loose and dense states.

$$\text{Relative density} = \frac{e_L - e}{e_L - e_D} \times 100 \text{ in percent} \ldots . 2.$$

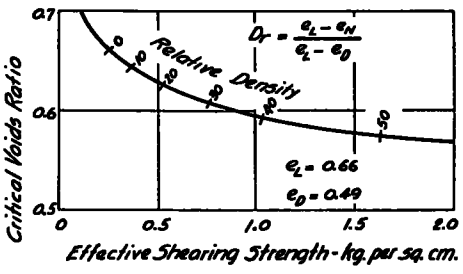
This curve represents the effective values the shearing strength can assume for each voids ratio or relative density for a critical condition during a flow failure. This curve has the appearance and many of the characteristics of the pressure-voids ratio for compression of the material, but is steeper because of the volume-change adjustments during shearing.

These relations are important because the finer-grained the material is, the greater is the probability that it has been deposited above the critical voids ratio. Casagrande (3) points out that nature deposits such materials almost without exception above the critical voids ratio. The failure of such deposits under heavy embankment loads depends on whether a flow failure condition can somehow develop, a sudden disturbance or a large foundation settlement playing the important rôle.

For embankments the Franklin Falls field compaction tests (10) (3) have demonstrated that there is no difficulty in obtaining densities greater than the critical density in cohesionless soils, even in the most heavily loaded portions of embankments of moderate height, by means of compaction equipment now used. Foundation materials of very loose character at Franklin Falls dam have recently been compacted successfully by means of explosives over a large area. The shearing strength of a well compacted mass can not be reduced by a sudden disturbance, but rather the soil mass will tend to have a greater shearing strength under such conditions, which increases the margin of safety against many of the uncertainties, particularly



(a) Approximate Volume-Change Relation During Shearing



(b) Approximate Relation Between Critical Voids Ratio and Effective Shearing Strength

Figure 2. Approximate Critical Density Relations for Standard Ottawa Sand

as to the true stress conditions in an earth embankment about which little is actually known. On the other hand flattening the slopes, which is the usual method resorted to, does not necessarily mean increased safety for the embankment.

COHESIVE SOILS—REMOLDED

In cohesive clayey soils and in very fine, relatively impermeable cohesionless silts, the problem of determining the shearing characteristics is more compli-

cated. The consolidation characteristics of the material must be known before one can analyze a situation and attempt an estimate of the limits within which the percentage of the total applied stresses that are effective in mobilizing shearing

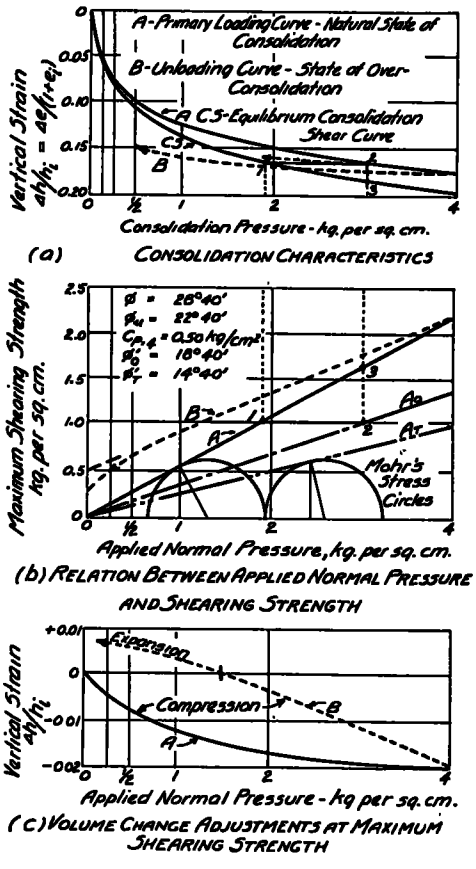


Figure 3. Shearing Characteristics of Remolded Detroit Clay

strength may lie. The permeability of these materials is so low that at ordinary rates of shearing the volume-change adjustments and full development of effective stresses can not take place. Hence a considerable proportion of the total stress may be carried by neutral stress in the pore water.

The results of direct-shear tests on remolded materials, for which conditions can be fairly well controlled, give us some clues on the behavior characteristics in the undisturbed state. The consolidation characteristics of a Detroit clay are shown in Fig. 3a by a typical primary loading curve—*A*, which represents a natural state of consolidation. Specimens of remolded clay at the liquid limit were completely consolidated under a series of normal pressures, which increased to the desired ultimate value for the shearing test with complete consolidation between each load increment, as in the regular consolidation test. It was found that if any other procedure was followed in pre-consolidating the specimens, the curves differed from those in a natural state of consolidation. For the unloading branch—*B*, representing a state of over-consolidation, all specimens in this series were completely consolidated under 4 kg. per sq. cm. and then the pressure was reduced in increments with complete expansion between each to the desired normal pressure for the shearing test.

The relation between the maximum shearing strength and the applied normal pressure, which gives the shearing characteristics of the material, is shown in Fig. 3b. The primary loading curve—*A*, in which the sample is sheared under the same normal pressure to which it was pre-consolidated, is a straight line through the origin. This means that a cohesive plastic remolded soil in a natural state of consolidation starting from the liquid limit, behaves essentially the same as a cohesionless material and therefore Coulomb's simple law expressed by Eq. 1 for cohesionless materials applies. In the slow shear test of about 24 hours duration, curve—*A*, the total applied normal stresses are effective stresses in mobilizing shearing strength, giving an angle of friction of 28° 40′ for the Detroit clay.

The curve of volume-change adjustments at maximum shearing strength in Figure 3c shows that for the primary loading curve, additional consolidation

takes place during shearing under all normal pressures. In the slow shear test the volume-change requirements in shear are satisfied and effective stresses are fully developed. In a rapid shear test, excess hydrostatic or neutral stresses would be induced in the pore water by the tendency for consolidation, which could not be dissipated during the shearing test by the escape of water, and therefore applied stresses would not be effective stresses.

By applying the ultimate volume-change adjustments for the slow shear test of Figure 3c to the primary loading branch of the consolidation curve—*A* of Figure 3a, an equilibrium consolidation-shear curve—*CS* is obtained for which applied total normal stresses are effective stresses. At point—*2* on the primary consolidation curve—*A* of Figure 3a the material is initially fully consolidated under the given normal pressure—p_2. When the shearing stress is increased to the maximum value in a slow shear test, further consolidation takes place to point—*3*, which lies on the equilibrium consolidation-shear curve—*CS*. A corresponding point—*3* is found on curve—*A* of Figure 3b for the slow shear test, which gives the maximum shearing strength for the given normal pressure—p_2. This point lies on the curve defining the true angle of friction.

Consider for a moment the conditions in the soil during shearing if no drainage is permitted, that is, if the specimen is sealed. There are difficulties in obtaining the curve for the angle of friction for zero volume-change by the usual quick shear test method, because, if it is run too rapidly, viscous effects make the results too high; and if run too slowly, some consolidation takes place, and the results are again too high. The direct-shear test method also gives results too high as compared with those of the tri-axial compression test, because of differences in the way the material reacts under test conditions. The material was initially consolidated at point—*2* under the normal pressure—p_2. When the shearing stress

TABLE 1

DETROIT CLAY

Sample—p-1

Remolded and tested at the liquid limit.

Liquid limit	30.0
Plasticity index	17.0
Specific gravity	2.70

CONSOLIDATION CHARACTERISTICS

Initial voids ratio at start of test, e_i	0.80
Voids ratio at pressure, p' of one kg. per sq. cm., e_v	0.57
Compression index, (slope of semi-log curve), c_v	0.059
Coeff. of consolidation, (½ to 1 kg.), c_c	0.015 cm.²/min.
Coeff. of compressibility, (½ to 1 kg.), c_a	0.28 1/kg. per sq. cm.
Coeff. of permeability, (½ to 1 kg.), k	8.0×10^{-6} cm./min.

SHEARING CHARACTERISTICS

		Angle of friction		Shear strength kg. per sq. cm.	
Primary loading curve—*A*	ϕ	28°	40′		
Unloading curve—*B*	ϕ_u	22°	40′	$p_N = 0$	0.25
Zero drainage, curve—A_o estimated	ϕ'_o	18°	40′	Cohesion	0.50
Zero drainage, curve—A_T Mohr's stress circles	ϕ'_T	14°	00′		

is increased gradually in a test where drainage is prevented, there is still the tendency for volume decrease, which is now restricted. Therefore excess hydrostatic or neutral stresses are induced in the pore water, which now carry part of the original pre-consolidated pressure—p_2. The normal pressure is no longer effective either as a consolidation pressure or in mobilizing shearing strength. Since the effective pressure on the grain structure of the soil is reduced, the voids ratio tends to adjust itself to the smaller pressure by expanding along the flat expansion curve dotted through point—*2*. But this expansion is also restricted, because the sample is sealed. The tendency does have the effect, however, of reducing the magnitude of the neutral stress in the pore water, and the equilibrium conditions are shifted back somewhat. Under these conditions the material has tended both to expand and consolidate along the horizontal line from point—*2* toward point—*1*, which, at the maximum shearing strength, is possibly located on the equilibrium *CS* curve. If this conclusion is substantially correct, the two points—*1* and *2* would be interrelated in such a way that they also represent the conditions, *1* when effective stresses are considered, and *2* when total stresses are considered under conditions of zero drainage. A corresponding point—*1* is obtained on curve—*A* in Figure 3b from the slow shear test in which the pressure—p_1 was the effective normal pressure throughout the test, as well as at the maximum shearing strength. A point—*2* on a curve—A_0, defined by the intersection of a vertical line through point—*2* of Figure 3a and a horizontal line through point—*1* of Figure 3b, would represent the conditions for zero drainage, in which the pressure—p_1 is the effective stress, and the pressure—p_2 is the total stress. Corresponding points for other horizontal lines in Figure 3a fall approximately on curve—A_0. The relations depend on how accurately the original consolidation curve and the equilibrium consolidation-shear curve—*CS* were determined. If these conclusions are substantially correct, the pressure—p_1, corresponding to pressure—p_2, as defined by a horizontal line cutting the two curves, may be considered to be the *equivalent effective pressure* acting at the time of maximum shearing stress during the test without drainage. The pressure—p_2 is the total pressure and the neutral stress—$u=(p_2-p_1)$. The shearing strength for the two points becomes—

$$S=p_1 \tan \phi=p_2 \tan \phi_0'=(p_2-u) \tan \phi$$

where ϕ is the value of the angle of friction—28° 40′, obtained by the slow shear test, and ϕ_0' is the apparent angle of friction for zero drainage, 18° 40′.

The value of the apparent angle of friction—ϕ'_0, obtained by direct-shear test methods is greater than the theoretical value of 14° 0′ obtained by consideration of total and effective stresses in the tri-axial compression test, which is about half of the true angle of friction. This is due to the essential difference in the way the soil reacts to the conditions which obtain in the direct-shear and the tri-axial compression tests. The Mohr stress circles for the direct-shear test intersect, while in the tri-axial compression test they are theoretically tangent to each other, but actually separated slightly, which is a phenomena resulting from the lateral bulging effects under vertical loading during the test.

It therefore appears that the direct-shear test, for the case of zero drainage, is influenced by inherent secondary effects which does not permit accurate determination of the angle of friction or of effective stresses. But for the slow shear and complete drainage and volume-change adjustments during shearing the angle of friction should be in substantial agreement with the true angle of friction

obtained by the tri-axial compression test.

Theoretical considerations of the tri-axial compression test, however provide a means of estimating the approximate percentage of the total applied stresses, which are effective in mobilizing shearing strength for the extreme case of zero drainage as a limiting condition, as illustrated by the Mohr stress circles in Fig. 3b. Such a condition is almost sure to obtain in a large mass of very impermeable clay, where drainage and consolidation under a superimposed load takes place with extreme slowness.

Considering the behavior characteristics of the material in a state of overconsolidation, the unloading curve—*B* in Figure 3b for shearing strength *vs.* normal pressure is characteristic just as the unloading or expansion curve—*B* of Figure 3a is characteristic for consolidation, representing a part of the hysteresis loop of the loading cycle. Curve—*B* is considerably flatter than the primary loading branch, but is linear over much of its length, giving an angle of friction ϕ_u for the state of overconsolidation of 22° 40′. Under the lighter normal pressures it curves downward and intersects the shearing strength axis at a positive value for p_N equal to zero of 0.5. It has been the usual practice to consider the intercept for the linear portion of the curve produced to intersection with the vertical axis as the cohesion. The particular value of the cohesion obtained depends first of all upon the maximum pressure to which the material has been pre-consolidated, and second on the moisture content at the particular stage of unloading as defined by the expansion curve for the consolidation test. This so-called cohesion represents a residual shearing resistance that remains from the maximum direct loading. It is apparent that cohesion is merely a phenomena of the hysteresis effect of a loading and unloading cycle. The values of cohesion and angle of friction so obtained are not soil constants, but vary widely with the method of testing and the consolidation characteristics of the material. This is one of the most controversial and least understood matters in soil mechanics.

An examination of the volume-change relations in Figure 3c for maximum shearing strength show that in a state of overconsolidation the compression during shearing decreases with a decrease in the normal pressure until a critical condition or voids ratio is reached at which the volume-change is zero. For still lighter normal pressures a considerable volume expansion or swelling occurs, as might be expected from unloading phenomena. In a slow shear test these volume-changes have time to take place and the sample absorbs water, where expansion occurs. But in a rapid test negative hydrostatic stresses are induced, and tension in the water now contributes to the shearing strength of the soil by temporarily increasing the effective stresses above the applied stresses. Hence the value of cohesion obtained by means of a slow shear test represents the ultimate value that can be counted upon, a value which is considerably smaller than that for the rapid test.

In rapid shearing the unloading curve —*B* for the slow shear test is in effect rotated about the point of zero volume change so that for pressures less than the critical value the rapid shear test gives values which are too high, and for greater pressures gives values which are too low. Thus the cohesion is higher and the angle of friction is lower than the values for the slow shear test. Only for the unloading curve obtained by a slow shear test are the conditions fully known so that Coulomb's more general law can be applied—

$$S = C_p + p_N \tan \phi_u \quad \ldots\ldots\ldots 3.$$

where C_p is the cohesion obtained for a maximum overconsolidation pressure—*p*, and is defined by the linear portion of the unloading curve produced to inter-

section with the shearing stress axis; and ϕ_u is the angle of friction obtained for the unloading cycle. The value of the cohesion depends upon the maximum pressure to which the sample has been consolidated, the overconsolidation pressure and the moisture content, as deter-

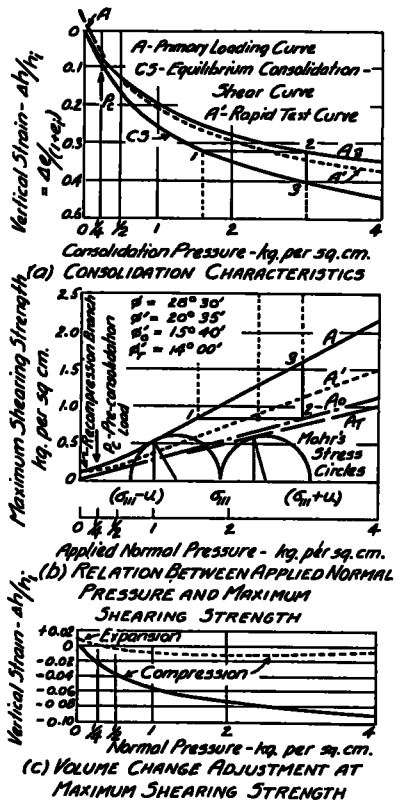


Figure 4. Shearing Characteristics of Flushing Meadow Silt-Clay

mined by the expansion characteristics of the material. The angle of friction for the unloading cycle appears to be independent of the overconsolidation pressure, but does depend upon the consolidation and expansion characteristics of the material.

UNDISTURBED COHESIVE SOILS—NATURAL STATE OF CONSOLIDATION

The consolidation and shearing characteristics of a typical sample of the Flushing Meadows, Long Island, soils are given in Figure 4. The material in the ground was in a natural state of consolidation under the weight of the overburden. In order to utilize the results of the tests we must separate the curves of Figure 4 into two parts. From the loading cycle relations for remolded materials in Figure 3 we recognize the first portion of the curves of consolidation and shear to be the hysteresis loops of the reloading cycle. This is a consequence of the removal of the sample from the ground and the reduction of the natural stresses, the pre-consolidation load as defined by Casagrande (11), to zero, with the accompanying swelling of the material. This reloading cycle does not represent actual condition in the ground, but merely the test phenomena. For the second part of the curves we observe that after the pre-consolidation load has been reached the curve becomes tangent to and practically coincides with the primary loading branch of the soil. The curve above the pre-consolidation load now represents approximately the conditions which actually obtain in the ground. It is characteristic for the primary loading branch—*A* of the curve in Figure 4b for a natural state of consolidation to be a straight line passing through the origin. Again Coulomb's simple law for cohesionless materials as expressed by Equation 1 applies. It is evident that only for pressures equal to or greater than the pre-consolidation load are test conditions representative of natural conditions or of subsequent conditions as the material is subjected to foundation or embankment loads.

The volume-change adjustments during shearing are given in Figure 4c. Applying these to the primary consolidation curve—*A* of Figure 4a, the equilibrium consolidation-shear curve—*CS* is obtained for the slow shear test. Curve—*A'* represents the conditions for the usual rapid shear test in which applied stresses are not effective stresses.

Applying the notions of an equivalent effective pressure, the horizontal lines in Figure 4a and 4b define corresponding points—*1* and *2*, which indicate that for the direct-shear test the applied stresses were only $p_1/p_2 \times 100$ or 53 per cent effective in mobilizing shearing strength in the rapid shear test. The theoretical percentage of effectiveness for zero drainage, obtained from Mohr's stress circles is 48.

The consolidation and shearing characteristics of samples 9-4 and 9-5 are given in Table 2.

For these soils the "quick" shear test under zero normal pressure or a simple unconfined compression test will provide an estimate of the shearing strength of the material which is somewhat lower than the shearing strength at the pre-consolidation load. The results are therefore on the safe side and the quick shear test or unconfined compression test could be used as rapid routine tests to obtain information on the shearing strength of the material for filling and grading operations in the development of the site for the World's Fair. However for stable slopes and control of deeper subsurface lateral movements in the deposit, and for estimating lateral pressures, etc., the shearing strength depends upon the effective stresses acting in the soil. Because of the very slow rate of consolidation of the material and of the thickness of the deposit, very little increase in shearing strength can be counted upon at deeper levels due to the filling and grading of the Meadows. Therefore the increased

TABLE 2

FLUSHING MEADOWS SILT-CLAY

Samples—9-4 and 9-5, Bridge C, Horace Harding Boulevard. Flushing Meadows, L. I.

Liquid limit	100 and 93
Plasticity index	44 and 42
Specific gravity	2.73

CONSOLIDATION CHARACTERISTICS

Depth of samples	42 ft. and 52 ft.
Overburden pressure	0.55 and 0.60 kg. per sq. cm.
Pre-consolidation load, p_c	0.25 kg. per sq. cm.
Voids ratio at pre-consolidation load, e_c	2.96
Initial voids ratio at start of test, e_i	3.00
Voids ratio at pressure, p' of one kg. per sq. cm., e_v	2.17
Compression index (slope of semi-log curve), c_v	0.36
Coeff. of consolidation, ($\frac{1}{4}$ to $\frac{1}{2}$ kg.), c_c	0.0018 cm.^{2} per min.
Coeff. of compressibility, ($\frac{1}{4}$ to $\frac{1}{2}$ kg.), c_a	1.44 1/kg. per cm.^{2}
Coeff. of permeability, ($\frac{1}{4}$ to $\frac{1}{2}$ kg.), k	8.8×10^{-6} cm. per min.

SHEARING CHARACTERISTICS

	Slow shear test		Rapid shear test	
	Angle of friction ϕ	Shear strength kg. per cm.^{2}	Angle of friction ϕ'	Shear strength kg. per cm.^{2}
Primary loading curve—A	28° 30′	....	A' 20° 35′	....
Estimated percentage effectiveness of p_N	100%	....	67%	....
Shear strength, $p_N = 0$	.....	0.098	.....	0.092
Shear strength at pre-consolidation load	.....	0.15	.....	0.12
Zero drainage, curve—A_0 estimated	15° 40′		percentage effectiveness of p_N	53
Zero drainage, curve—A_T Mohr's stress circles	14° 00′			48.5

pressures are not effective in mobilizing shearing strength. The conditions are approximately given by the Mohr stress circles in Figure 4b, and the original overburden pressure is only approximately 48 percent effective.

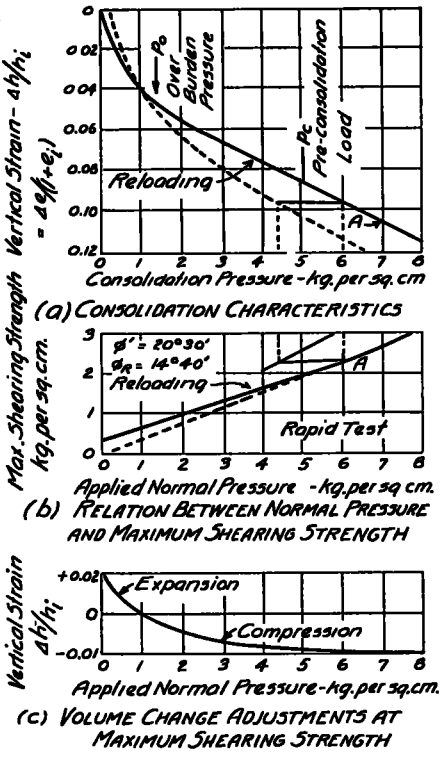


Figure 5. Shearing Characteristics of Passaic Valley Glacial Clay

UNDISTURBED COHESIVE SOILS—STATE OF OVERCONSOLIDATION

A different condition is observed in a clay deposit, which is in a natural state of overconsolidaiton. Quite frequently a clay deposit has been subjected to a greater overburden pressure sometime in its geologic history than now exists, for example when a part of the overburden has been removed by erosion. Such a condition is illustrated in Figure 5 for a glacial clay from the Passaic River Valley, N. J. The consolidation and shearing characteristics are given in Table 3.

In this case we are operating on a reloading branch of the curve—R, and conditions approximate those in the natural state for pressures equal to the overburden pressure and for pressures greater than the overburden pressure due to subsequent loadings from proposed structures. In this case the shearing strength can be expressed by Equation 2—$S = 0.28 + p_N \tan 14° 40'$, where p_N represents the total normal stress in this case for a fairly rapid shearing test. This relation is only applicable, however, for pressures greater than the overburden pressure and less than the pre-consolidation load, and further provided that the percentage effectiveness of the applied stresses are the same; which means that very little is known about the shearing strength in this case, because only a rapid shearing test was made.

ARTIFICIALLY COMPACTED COHESIVE SOILS

The consolidation and shearing characteristics of a cohesive soil, compacted by the standard Proctor method of compaction are given in Table 4 and shown in Figure 6. For pressures less than one kilogram per sq. cm. the material tends to expand during shearing and hence in a rapid test the effective pressures and the shearing strength are temporarily increased. For pressures greater than this the material tends to compress during shearing and part of the applied normal stresses are not effective but carried by neutral stress in the pore water. An artificially compacted soil is subjected to internal capillary pressures. These internal capillary pressures and the original equivalent compacting pressure produce a state similar to that of overconsolidation under an equivalent compacting maximum pressure. In an earth dam the portion of the initial shearing strength contributed by the internal capillary pressures will ultimately be reduced to zero by complete saturation when the entrapped air passes into solution. There-

TABLE 3

GLACIAL CLAY, PASSAIC VALLEY, N. J.

Sample—5-b. Varved clay with silt partings.

Liquid limit	53.8
Plasticity index	24.8
Specific gravity	2.70

CONSOLIDATION CHARACTERISTICS

Depth of sample	40 ft.
Overburden pressure	1.40 kg. per sq. cm.
Voids ratio at overburden pressure	1.125
Pre-consolidation load, p_c	5.0 kg. per sq. cm.
Voids ratio at pre-consolidation load, e_c	1.05
Initial voids ratio at start of test, e_i	1.20
Voids ratio at pressure, p' of one kg. per sq. cm., e_v	1.32
Compression index (slope of semi-log curve), c_v	0.17
Coeff. of consolidation, (1 to 2 kg.), c_c	0.045 cm.^{2} per min.
Coeff. of compressibility, (1 to 2 kg.), c_a	0.036 1/kg. per cm.^{2}
Coeff. of permeability, (1 to 2 kg.), k	3.6×10^{-6} cm. per min.

SHEARING CHARACTERISTICS

Rapid test.

Primary loading curve, produced—A	ϕ' 20° 30′
Reloading curve, R	ϕ_R' 14° 40′
Shear strength, $p_N = 0$	0.28 kg. per cm.^{2}
Shear strength at overburden	0.75 kg. per cm.^{2}

TABLE 4

COHESIVE EMBANKMENT MATERIAL

Sample—Test pit sample, No. 8. Passaic River Valley, N. J.

Liquid limit	20.8
Plasticity index	17.8
Specific gravity	2.70

Maximum density, Proctor compaction test	125.6 lb. per cu. ft.
Optimum moisture content, w_o	11.6 percent.

Samples tested at 110 percent of optimum and compacted to standard degree.

CONSOLIDATION CHARACTERISTICS

Initial voids ratio, at start of test, e_i	0.343
Voids ratio at pressure, p' of one kg. per sq. cm., e_v	0.308
Compression index, (slope of semi-log curve), c_v	0.014
Coeff. of consolidation, ($\frac{1}{2}$ to 1 kg.), c_c	0.25 cm.^{2} per min.
Coeff. of compressibility, ($\frac{1}{2}$ to 1 kg.), c_a	0.022 1/kg. per cm.^{2}
Coeff. of permeability, ($\frac{1}{2}$ to 1 kg.), k	7.0×10^{-6} cm. per min.

SHEARING CHARACTERISTICS

Slow shear test.

Angle of friction	39° 30′
Cohesion	0.55 kg. per sq. cm.
Shear strength at p_N	0.40 kg. per sq. cm.

fore it is on the unsafe side to count upon the cohesion supplied by internal capillary pressures as permanently contributing to the stability of the structure. Casagrande (3) has emphasized that in stability analyses of earth dams only the

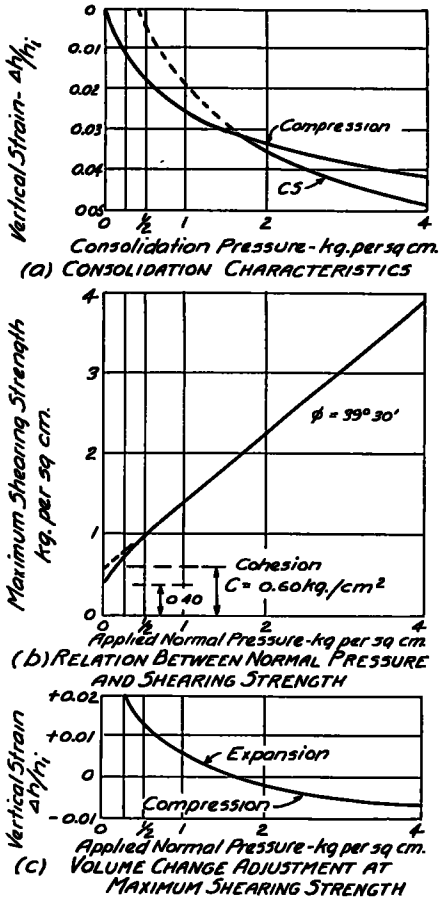


Figure 6. Shearing Characteristics of Compacted Cohesive Soil

shearing strength which can be counted upon for certainty after a number of years should be used, namely the shearing strength mobilized by permanently effective stresses due to the weight of the earth in the structure, which will allow the true cohesion existing after a number of years to provide an additional factor of safety against some of the uncertainties inherent in design. This means an attempt must be made to estimate the percentages of the total pressures due to the weight of the material itself, that are effective in mobilizing shearing resistance for certain most dangerous conditions.

1. Upstream and downstream slopes. During and just subsequent to completion of the structure before the interior portions have reached complete consolidation and equilibrium under the weight of the overlying earth.
2. The upstream slope during a rapid drawdown.
3. The downstream slope under seepage forces.
4. The foundations, when the most dangerous sliding curve passes into a weaker underlying layer of material, and this material has not completely consolidated and come to equilibrium under the weight of the structure.

The greatest uncertainties at the present time in the design of earth structures are due to the lack of accurate knowledge, on the true stress distribution in the embankment itself and how these stresses change with time; on the stresses induced in the foundations, and how they are affected by the subsurface conditions; and how the embankment and the foundations adjust themselves to these stresses by settlements and lateral movements.

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LIMIT DESIGN OF FOUNDATIONS AND EMBANKMENTS

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SYNOPSIS

Procedures based on limit design have been in use for some years in soil mechanics. The main contribution of this paper which outlines the basic features of designing on the limit principle, is a proposed method for combining failure criteria and a factor of safety. It is presented with the belief that before using any method of analyzing earth structures, it should be emphasized that accuracies of results can be no higher than the accuracy with which the strength properties are known and that any factor of safety which is used must cover numerous items of uncertainty.

An important factor in the setting up of any method of design is the choice of what the criterion of failure shall be. Two criteria have often been proposed and have been applied in designs of structures, using from the commonest building materials,—steel, concrete and wood, to the more natural materials,—rock and soil. The choice between these two criteria may be worded in this way: shall failure be associated with the condition wherein the most dangerously stressed point just reaches its limit of strength, or shall it be considered that failure has not occurred until the point which first attains the plastic state has undergone further strain, under constant or decreased strength, until all reserve of strength of surrounding points has been expended? The first criterion involves only the limiting value of the stress, the second brings in relationships between stresses and strains. When a design uses this second failure criterion it is spoken of as a "Limit Design."

Much recent discussion of the relative merits of the two criteria has taken place. A stimulating paper by J. A. Van den Broek[1] has recently presented a strong argument for Limit Design in steel structures and has been the cause of considerable discussion.

In the field of soil mechanics it has repeatedly been emphasized that stress-strain relationships are of outstanding importance as soil properties. Much research has been devoted to determinations of shearing strength of soils and in spite of controversial points of view on numerous details, it is generally admitted that real progress has been made in the general knowledge of the stresses which soils can withstand. However, in addition, steadily increasing emphasis has been placed on the importance of the strains associated with these stresses. It must be recognized that a clear understanding of strength is impossible without thorough knowledge of the associated volume changes and shearing strains.

To express the shearing strength of soils, the empirical law of Coulomb is commonly used. It is written

$$s = c + p \tan \phi$$

wherein s is the shearing strength, c the cohesion, p the normal intergranular stress on the incipient shearing surface and ϕ the angle of internal friction of the soil. The soil properties c and ϕ must not be spoken of as constants as they vary widely for the different conditions under which shear may occur. However, for any given homogeneous embankment under a given set of conditions, there probably are values for c and ϕ which will represent the situation with reasonable accuracy. Typical curves of shearing stress against shearing strain are shown in Figure 1. Loosely deposited sands and soft clays have stress-strain

[1] *Proceedings* A.S.C.E., February 1939.

curves of the shape of curve I. Dense sands and many clays which in their natural state show appreciable structural strength, reach a maximum shearing strength at some certain shearing strain, but undergo a decrease in strength at larger strain, as shown by curve II. Unless otherwise stated it will be assumed that the curve under consideration is of the shape of curve I.

As an example of the type of analysis wherein all stresses are kept below the strength of the material, the case of the earth dam of Figure 2, founded on a deep deposit of highly cohesive clay will be used. This analysis was first presented by L. Jürgenson.[2] To simplify the analysis it is assumed that the second term on the right hand side of Coulomb's expression is negligible or that the shearing strength is constant throughout the clay at 1000 lb. per sq. ft.; that the embankment of approximately triangular shape produces a triangular distribution of load on the surface of the clay stratum and that the stresses produced by this loading in the clay stratum are equal to those which would occur if the clay were a truly elastic mass. Computations based on the theory of elasticity give the stresses shown in Figure 2. It is seen that the maximum shearing stress of 920 lb. per sq. ft. is slightly lower than the shearing strength of 1000 lb. per sq. ft.

In order to express the degree of safety, some form of "Factor of Safety" must be introduced. In determinate trusses, the use of a safety factor leads to little confusion. However, the greater the degree to which a structure is indeterminate, the harder it is to obtain a clear understanding of the degree of safety and the more obscure becomes the meaning of "Factor of Safety." If there are one-hundred possible ways in which a structure as a whole could fail, then at least one hundred different factors of safety could be described. Yet the use of a "Factor of Safety" is an engineering necessity. Therefore, whenever the term is used, if any possibility of confusion exists it is evident that a careful description must be given.

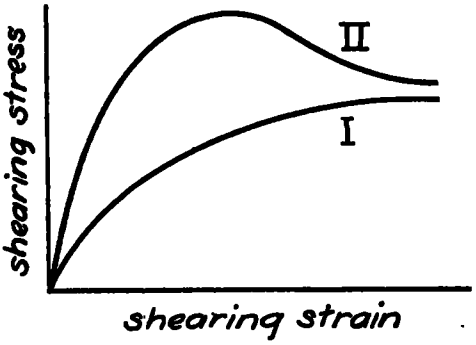


Figure 1

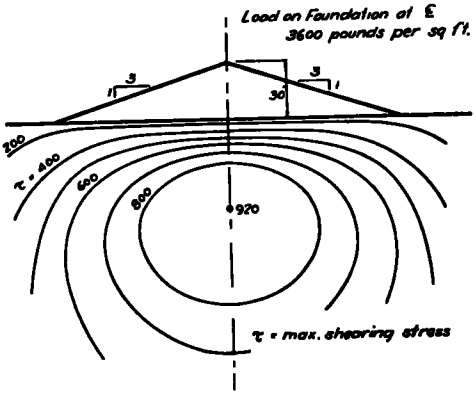


Figure 2

In any soil stability study, it is a statically indeterminate problem that is dealt with, and the degree to which it is indeterminate is itself indeterminate. Therefore, any factor of safety, before being used, must be fully described. The scheme in common use for cases such as that above is to use the ratio between shearing strength and maximum shearing stress; in this case $1000 \div 920 = 1.1$.

[2] "The Application of Theories of Elasticity and Plasticity to Foundation Problems" (Fig. 9) by L. Jürgenson, *Journal*, Boston Society of Civil Engineers, July 1934, Vol. XXI, No. 3.

The value of 1.1 is sometimes spoken of as the *point* factor of safety. It could also be called the factor of safety against the beginning of a plastic condition.

It should be realized that in any stability analysis on soils there are many variations from the idealized conditions which are assumed and which the factor of safety must absorb. Therefore, two significant figures for the above factor are sufficient in view of the low degree of probable accuracy.

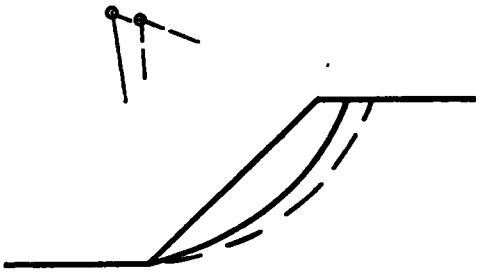


Figure 3

If the dam of the above example were gradually built to steeper slopes on the same base width, the stress would increase in proportion to the load. After a point factor of safety of unity is attained a plastic zone would develop and further load would cause this zone to expand until, at the limit load, the plastic zone would reach ground surface at some point. For soils represented by curves of form I of Figure 1, the limit load would be considerably larger than the load causing the most highly stressed point to just reach its limit of strength. For a soil with a stress strain relationship similar to that of curve II the limit load would exceed the load causing point failure somewhat. However, this excess might be very small if the stress-strain curve shows a large stress drop after reaching the maximum value.

A limitation of the above analysis is that it involves the assumption of elastic material and can be used only for sections where the elastic theory or photoelastic studies are capable of supplying the stress pattern. In soil mechanics studies it has been used only for foundation analyses although its use has been suggested for embankments.[3] Van den Broek claims that the point type of analysis is not as rational as the limit design method. The writer accepts this contention but believes that in soil stability studies the outstanding disadvantage of the point scheme as illustrated above is that results by it can only be as accurate as the degree to which soil behaves like a truly elastic mass.

Limit design in soils wherein the complete shearing strength is developed over an entire failure surface or throughout a failure zone, can best be illustrated by an analysis of a simple slope.

Field investigations of many slope failures in Sweden have demonstrated that if a slope such as that shown in Figure 3 were to fail, rupture would occur along a surface which can be approximated with reasonable accuracy by a circular arc. The "Circular Arc Method," which originated from this evidence, was developed by W. Fellenius.[4] A general solution based on this method for a simple slope of homogeneous soil such as shown in Figure 3 has been developed by the writer.[5] The procedure in brief is to locate in some way that circular arc which is most liable to rupture, assume all shearing strength is being utilized at every point along the arc and analyze the conditions which must exist if the mass is just at the point of failure. The analysis consists of expressing the equilibrium of moments about the center of the circle. The solution is of the form

[3] See Discussion on Soil Mechanics Symposium by T. T. Knappen *Transactions* A.S.C.E., Vol. 103, p. 1451.

[4] "Erdstatisch Berechnungen mit Reibung und Kohäsion," W. Fellenius, Berlin 1927.

[5] "Stability of Earth Slopes," D. W. Taylor, *Journal*, Boston Society of Civil Engineers, July 1937, Vol. XXIV, No. 3.

$c = \gamma HK$, wherein c is the cohesion required to just maintain equilibrium, γ the unit weight of the soil, H the vertical height of the slope and K a term which depends only on the slope angle and the angle of internal friction. The expression $\frac{c}{\gamma H} = K$ is a dimensionless constant to which the writer has given the designation of "Stability Number." For any given inclination of slope, in a homogeneous material which has an angle of internal friction ϕ, the stability number may be obtained from Figure 4.

In Figure 3 the slope is 45 deg. and the friction angle will be assumed to be 20 deg. From Figure 4 the stability number is 0.062 and any combination of values of c, γ and H which give $\frac{c}{\gamma H} = .062$ will describe a condition which is just at the point of failure. The example which will be used is c = 500 lb. per sq. ft., γ = 120 lb. per cu. ft. and H = 67 ft. This example of limit design illustrates the commonest type of stability analysis of earth structures and has been in use since the earliest days of the branch of engineering we now call soil mechanics.

Before the above analysis may be used for an actual engineering problem some rational method of expressing a factor of safety must be determined upon. At first glance it might appear that there would be a factor of safety of 2 if, instead of 120, the unit weight were to be 60 lb. per sq. ft., a reasonable value if the slope were the bank of a canal and completely submerged below free water surface. Similarly if the height were but 33.5 ft. instead of 67 ft. or again if the cohesion were 1000 instead of 500 lb. per sq. ft., the actual stability number would be twice the limit value. It is seen that such changes are equivalent to decreasing the loading by half, but reference to Coulomb's law shows that any decrease in loading also causes a decrease in the strength of the soil. Thus the gain in safety is not so large as would first appear. For case of a crib carrying a 1000-lb. load, an extreme example of this point, the resting on a rock slope which is so steep that the crib is just at the point of slipping, may be considered. The factor of safety against slip is just unity. Reducing the load will not improve this

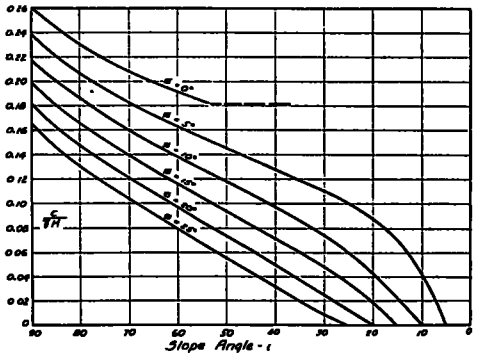


Figure 4. Chart for Stability No. $\frac{c}{\gamma H}$

factor as slip will still be incipient so long as the coefficient of friction is a constant. However, in the simple slope under consideration, if by any one of the three ways mentioned the actual stability number were to be doubled, the factor of safety that would result is sometimes spoken of as 2. It is important that a better terminology be introduced for use here, such as "Factor of Safety with respect to Cohesion" or "Factor of Safety with respect to Height."

For cases such as the above the writer advocates a different approach using a "Factor of Safety with respect to Strength." If the value of this factor is $1\frac{1}{3}$, then the strength must be $1\frac{1}{3}$ times as great as is required to just avoid failure, or if all strength were to be divided by $1\frac{1}{3}$ a condition of failure, or limit condition, would result. Any actual case wherein the factor of safety with respect to strength is $1\frac{1}{3}$, is static-

ally indeterminate. However, if in the actual case the soil were to be replaced by a hypothetical soil with a strength equal to the actual strength divided by $1\frac{1}{3}$, a determinate case would result. The ideas are the same as those pointed out by Van den Broek in an example on rivet design.

Experience has shown that for practical and economic reasons, factors of safety with respect to strength in embankment analyses must frequently be limited to values of the order of 1.1 to 1.5. Again referring to Figure 3 and assuming a 45-deg. slope, a friction angle of 20 deg., a cohesion of 500 lb. per sq. ft. and a unit weight of 120 lb. per cu. ft. and introducing a factor of safety of $1\frac{1}{3}$ by using a working value of cohesion of $\frac{500}{1.33}=375$ lb. per sq. ft. and a working value of friction angle which has a tangent equal to $\frac{\tan 20}{1.33}$, or 15.2 deg., the allowable height may be determined as follows: The stability number for a 15.2-deg. friction angle is 0.082, and

$$0.082=\frac{c}{\gamma H}=\frac{375}{120H} \text{ whence } H=38 \text{ ft.}$$

Actually the most dangerous circle for a 15.2 degree friction angle is the dotted arc of Figure 3 while for 20 deg. it is the full line arc, but this need not be considered in the present discussion. Since the height which would just cause incipient failure was 67 ft., the factor of safety with respect to height for a 38 ft. embankment is 67/38 or $1\frac{3}{4}$; as has already been pointed out $1\frac{3}{4}$ may also be called a factor of safety with respect to cohesion. At the same time the factor of safety with respect to strength is $1\frac{1}{3}$. Failure of this embankment would occur if any one of the following changes should occur; height multiplied by $1\frac{3}{4}$; unit weight multiplied by $1\frac{3}{4}$; cohesion divided by $1\frac{3}{4}$; shearing strength divided by $1\frac{1}{3}$.

If a soil has a stress-strain curve similar to curve II of Figure 1, since the shearing strength is developed progressively, the maximum shear at the instant of complete failure will occur at the point on the failure arc where the shearing-strain is smallest. The average stress at this limiting condition falls somewhere between the maximum value of stress and the ultimate value, its location between these two limits being statically indeterminate. In fact a determination of the stresses would require complete knowledge of stress-strain relationships of the soil and of the strains developed all along the failure surface. Although the discussion given above mentions stress and strength frequently it does not do so at the expense of consideration of strain. Strains and their effect are completely covered and a true limit condition is under consideration when full shearing strength is assumed over the entire arc. In soils, strains are of such an involved nature that it would be difficult to talk in terms of them.

An outstanding example of limit design which is in common use by structural designers is that of retaining wall pressures. Dr. Karl von Terzaghi[6] proved by his experiments on the large model retaining wall at the Massachusetts Institute of Technology that the lateral pressure is very dependent on the amount of yield of the wall, or, what is the same, the amount of strain in the backfill. Sufficient yield away from the fill leads to a condition of complete summoning of shearing strength in the backfill and to a minimum pressure known as the active pressure. This is shown diagramatically by Figure 5. Similarly, sufficient wall displacement toward the fill leads to "passive" pressure. From this it may be seen that the pressure acting on a wall will be greater than the active pressure unless the wall is just barely strong enough to withstand active

[6] *Engineering News-Record*, Spring 1934, Vol. 112.

pressure. Thus any wall which has a factor of safety, resists greater than active pressure, yet designs are based either on active pressure theories or on rule of thumb methods which spring at least partly from active pressure conceptions. The saving feature is that if the pressure at any time happens to overtax the wall, the wall yields just enough to relieve the pressure to a value which no longer overtaxes it.

Many types of factor of safety have been proposed for use in retaining wall design. The writer can suggest no more rational approach than to divide the limit strength of each unit,—soil, wall material, etc.—by whatever factor of safety with respect to strength is reasonable for that unit. Then proceed to the setting up of a design wherein as many as possible of these working values of strengths are just attained and others are on the side of safety. Thus the design value for pressure on the wall would not be active pressure but would be the value the active pressure would assume in a hypothetical soil wherein the shearing strength equals the quotient of the strength of the true soil divided by a factor of safety with respect to strength.

A method of limit design for sheet pile bulkheads was suggested by Dr. Glennon Gilboy a few years ago in his graduate soil mechanics course at the Massachusetts Institute of Technology. By this scheme the piling is first figured as though driven only to the toe-hold required to just give sufficient passive pressure to prevent failure at the toe. For this case the wall pressures are determinate and the piling and tie-rods can be designed by conventional methods. Subsequently the piling is actually made longer by an amount sufficient to give a reasonable factor of safety against failure at the toe. Study of the interrelation between wall-yield and wall-pressure indicates that this added depth at the toe relieves the stresses in the tie-rods and piling and thus gives an indeterminate amount of added safety with respect to these items. The writer would like to suggest that here again the use of the hypothetical soil, with a strength which is expressed by the quotient of true strength and a relatively small safety factor with respect to strength, could be introduced in place of the first computations of the above

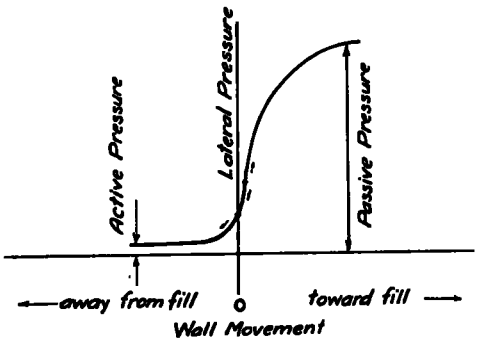


Figure 5

scheme. The resulting wall would have the desired factor of safety in every respect, including that against toe failure.

In conclusion, it must be admitted that rule of thumb methods are needed for simple routine designs. However, in important structures where a good visualization of the interrelated behavior of the numerous elements is important, the limit design for earth structures at least is the rational approach. In any analysis which involves criteria of failure some form of factor of safety must be used and the form which is chosen should be clearly defined. The writer has found that where structural elements have strengths which depend on the loading, the most satisfactory approach is to describe working values of strength as equal to the actual strength divided by a factor of safety with respect to strength; then to proceed with a limit design wherein these working values are treated as though they were the only strength in existence.

EFFECTS OF BASE COMPACTION ON MAINTENANCE COSTS AND PERFORMANCE

By J. J. Forrer

Maintenance Engineer, Virginia Department of Highways

SYNOPSIS

Emphasis in this paper is placed on the importance of proper compaction of highway embankments and the necessity of a certain time passing before the complete stabilization of the materials can be accomplished. In the desire to open a road as quickly as possible, this time element is often overlooked. The maintenance man frequently finds that the beautiful road turned over to him soon settles and breaks up, requiring repairs soon after completion. The psychological effects of seeing holes being drilled in new pavements to allow mudjacking operations to correct faulty compaction is undesirable. It is likewise expensive.

As the conditions under which embankments are placed can seldom be controlled to an ideal, uniform compaction cannot be secured by mechanical means alone. Time must be allowed for complete adjustment of the particles. Experience gained on maintenance work indicates that at least one year's time should elapse before placing a high type costly pavement. During such time a temporary surface such as gravel, crushed stone or cheap bituminous treatment should be applied.

The discussion is also true for the so-called lower type pavements. Even though these receive mechanical compaction, some time should be allowed for final adjustment to take place before they are subjected to the maximum traffic.

The object of this paper is to emphasize the importance of the proper compaction in highway construction work and the time element that is so necessary to bring about the complete and final stabilization of the materials used. The time element is often overlooked in the hurry to get the road opened to traffic.

The effects of faulty rapid construction bring about maintenance operations that are costly and at the best unsatisfactory. How often does the maintenance man find the beautiful road turned over to him, settling out of shape, breaking up, and needing repairs in less than a month after it is completed? All of the patching, mudjacking or other efforts, that are most costly, cannot bring a pavement that has settled out of shape, back to the condition it should have been in when completed. Naturally the psychological effect is bad to see maintenance crews drilling holes in a new pavement or performing other operations necessary to correct faulty compaction of materials in construction.

In the grading operations, it is seldom that conditions are ideal to bring about uniform or final compaction by mechanical means alone. In rolling a fill, if the materials are too wet, they will muck; if just moist, they will become tough and bridge over; and if too dry, they will break down into powder. Even if an effort is made to control the moisture content, as called for in some specifications, time must also be allowed for a complete adjustment of particles. From the experience gained with the maintenance of many miles of new pavement, it appears a year is the minimum time before a high type, costly pavement can be laid on a grade, with safety. During this period the grade should be thrown open to traffic and a temporary surface maintained with crushed stone, gravel or other materials. Dust on such a road could be held to a minimum by one of the dust palliatives in common use, or by a cheap bituminous surface treatment.

Such a procedure may be considered a step backward, but public opinion will soon be behind the engineer, who can

build a road that will stand up and take the traffic it is subjected to instead of one that starts to fail a month after it is completed.

The above argument has dealt only with the grade. The same principles that apply to the grade are only too true for the so-called lower type pavements. Surfaces made of sand-clay, soils, gravel or crushed stone must have time to compact thoroughly before they will carry their maximum capacities without failure. Even though assisted by mechanical means these types need time to take a final adjustment. Bituminous surface treatments on these lower types will surely fail if applied before traffic and weather conditions have been allowed to have their effect.

Again it is most disconcerting to the maintenance man, and at the same time puzzling to the public, to take over a new bituminous surface treated road that in a few months begins to fail. All who have had experience with the lower type roads have seen many first bituminous treatments fail within a year.

This could be prevented by allowing more time for the surfacing materials to cure and take their final compaction. Assisted by dust palliatives and the proper machining operations, roads constructed with sand-clay, soil or other low type materials will carry traffic most comfortably while the setting up process is in progress.

There are cases where the low type surfaces, that have been allowed time to consolidate, are carrying the same traffic as the higher types on the same road and at no more maintenance cost.

Optimum moisture content is usually defined as the amount of water necessary to be mixed with soil to produce the maximum compaction with a sheepsfoot roller or other mechanical means. This sounds simple enough, but it is doubted if many of the states are actually carrying this procedure to actual practice. The whole matter seems to be looked upon with suspicion and will be until

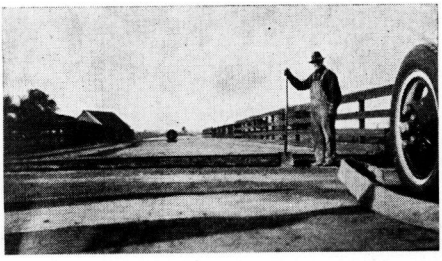


Figure 1. Settlement of a Concrete Pavement Slab Next to a Bridge

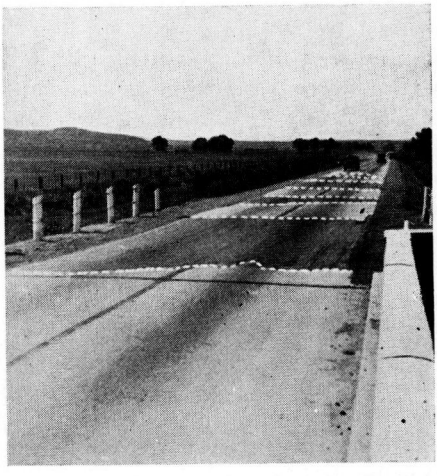


Figure 2. Settlement of a Concrete Pavement Slab Over a Low Fill

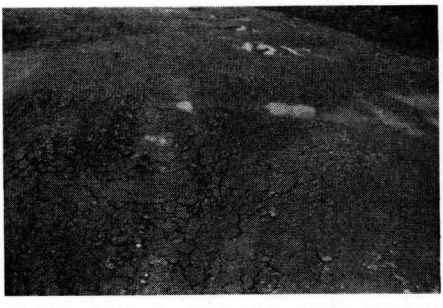


Figure 3. Failure of Bituminous Surface Over a Poorly Compacted Base and Fill

further knowledge on the subject is made general. Quoting from a leading engineer, who has had many years of experi-

ence, he has the following to say. "It is unfortunate indeed that so many engineers have only a slight knowledge concerning soil behavior and are little concerned about it. To many, 'dirt is dirt,' and their designs are usually made massive to take care of the worst anticipated conditions, with no thought or provision for possible modification of the soil itself." Further quoting from the same source, "Many bases have failed, not because of poor quality, but because they were not consolidated, and the surface placed thereon became distorted later as further compaction occurred under traffic."

Figures 1, 2 and 3 show the results of faulty compaction.

DISCUSSION ON BASE COMPACTION

MR. W. W. MACK, *Delaware Highway Department:* A pavement about 20 years old on a heavily traveled route—heavy truck traffic—began to show failure and we began to resurface it. It was noticeable that it was in good condition in some localities and in others it was in very bad condition. It was decided to patch it with concrete and resurface it with bituminous material and before proceeding with it cores were taken throughout the job and soil samples were taken underneath the pavement. The point of interest to me was that in 95 per cent of the samples the strength of the concrete varied in proportion to the soil classification. The soil classification ranged from A1 to A9 and in the A9 soil it was impossible to remove the core and in the A1 soil there was a good quality of concrete. It showed conclusively that the failure was not due to nonuniformity of concrete but rather to nonuniformity of subgrade material.

MR. W. H. ROOT, *Iowa Highway Commission:* I have always felt that the maintenance division of a highway department has two distinct functions: First, the maintenance organization is supposed to keep the highways repaired so that they are usable, and second—and probably of more importance—they are supposed to detect errors made in design and construction. Unless these errors are found and corrected we are not fulfilling our job. Speaking of specific instances we are beginning to have failures in concrete pavements built 10 or 12 years ago most of which are due to corner breaks occurring in cuts where we now know that the subgrade was unsatisfactory. We are not placing pavements on that type of subgrade any more, but it took us a long time to find that out. In that connection I would like to point out one other thing. I am not sure that those pavements in soft cuts would even have stood up if our loads had had a legal limit of 4 tons per wheel. In the weighing of vehicles, we are finding that one out of three vehicles violates our laws, not only in weight but in other respects. A good many are carrying 10 and 12 tons to the wheel. I think the roads are just beginning to get these big loads which break them down.

MR. W. R. MACATEE, *The Asphalt Institute:* Has there been any increase in the contact areas of the tires? Mr. Root indicates that there has been considerable increase in weight. How about the increase in contact area?

MR. C. N. CONNER, *Public Roads Administration:* A number of speakers at these sessions have very properly called attention to the importance of designing subgrades. By experiment and analysis we know something about loads and their bell-shaped distribution of stress through flexible pavements to the underlying subgrades. We know, also, that the intensity of pressure on subgrades under flexible pavements is greater than the unit pres-

sure under rigid pavements. However, there is no generally accepted rational formula for the design of subgrades or for flexible type surfaces.

Wherever soil mixtures under pavements have been designed and constructed to have a uniform and known bearing power sufficient to support the pavement and its live load without subgrade displacement, the life of the pavement has been increased and its maintenance cost reduced. Experience in California, Massachusetts, Michigan and elsewhere has shown that the cost of preparing a proper subgrade can be justified whether the pavement is of the rigid or the flexible type.

A practical and useful discussion of subgrade design and construction utilizing low-cost local soils may be found in a report submitted to the Design Department by Bernard Gray at the Joint Session of soils and Design on December 5.[1]

C. M. UPHAM, *Chairman:* Mr. Poulter, have you an opinion as to why some of these failures do not occur right away?

[1] "Present Design Practice and Construction Developments in Flexible Pavements," See page 175 this Volume.

MR. JOHN W. POULTER, *Koehring Machine Company:* The causes may start four or five years back or develop rather slowly depending on the amount and nature of the traffic. The action of traffic over the slab may cause a shifting of the water under it, more particularly at a joint or crack. This may cause a slight washing effect, very little at first and increasing in rate, very slowly, as traffic continues to use the pavement until enough subgrade material is washed out to allow the slab to break down.

MR. FORRER: To substantiate Mr. Poulter's statement, the condition we are finding in Virginia is this: that on our heavy traveled roads that were built of concrete—some of the newer pavement joints began to pump in less than a year's time. The amount of heavy traffic is a controlling factor. Pavements carrying a small amount of heavy traffic do not develop the pumping action as soon as pavements that are continually pounded by a large amount of heavy traffic. Subgrade treatment seems, for the time being at least, to be solving the problem of pumping action but even then if too much water is allowed to get under the pavement trouble can be expected.

CONTROL OF FROST HEAVE

By R. M. Rowat

Canadian Industries, Limited

SYNOPSIS

The author describes the use of salt in the prevention of frost heave on railroad tracks. Frost heave will not take place if the soil water in a susceptible area does not freeze. When salt dissolves in the water, the freezing point is lowered. As it is possible to determine the amount of salt required to lower the freezing point to any desired level, it is only necessary to know the minimum temperatures to be guarded against.

Several methods of applying the salt to the roadbed are discussed, but the important factor is to apply the salt early enough so that it may be thoroughly distributed over the susceptible area by the time freezing starts.

The author suggests the use of salt as an admixture in subbase courses of highways as a frost control measure with no particular reference to stabilization. Such a procedure has been followed in the construction of highways in Nova Scotia during the past two years. The addition of from 20 to 40 tons of salt per mile has been quite satisfactory as frost control.

My subject has to do with certain experiments on the control and prevention of track heaves caused by frost action carried out over the past few years by Canadian railways. Since the control of differential frost heave is a problem in both the railway and highway fields, the information gleaned in the one can be presented with the hope that it will be found interesting and perhaps useful.

It is undoubtedly true that frost heaves on tracks are a much more serious problem to the railways than frost heaves on highways are to highway engineers. Smooth track is not just something desirable from the viewpoints of passenger comfort and economical operation; it is an absolute essential to safe operation of modern high-speed trains. A small heave, that, on a highway might be but a minor nuisance to motor traffic, on a railway track becomes a positive menace calling for immediate correction lest human lives be endangered, to say nothing of potential property damage.

Therefore continuous inspection for track movement and prompt adjustment of same are amongst the most important winter duties of track maintenance crews. Adjustment is accomplished by "shimming" the rails up on an even and gradual approach to and departure from the heave site, using hardwood shims of varying thicknesses in the process. Thus a short heave, if it be of only a few inches, may necessitate "shimming" hundreds of feet of rail, and it will be obvious that a one-sided heave affecting but one side of a track, must call for equal "shimming" of both sides. You will not then be surprised to learn that millions of shims are used annually by our railroads, and that the cost of placing—and later removing them runs into very substantial figures.

The experiments discussed here were promoted by the salt industry and involve the use of salt. May I briefly outline the theoretical basis on which we worked.

It is unnecessary to discuss the etiology of frost heaves,—you are all familiar with their causes. Nor do I need to repeat that adequate drainage, or excavation and back-fill with porous material will in the majority of cases eliminate them. These are well known facts.

But, as these methods are not always applicable for topographical or economic reasons, a cheap chemical control method has some interest.

The theory is simply that frost heave

will not take place if the soil water in a susceptible area does not freeze. There will then be none of the expansive force of ice formation, no migration of water to ice particles in the soil, and no heave.

Now, according to the records of underground temperature gathered by the Meteorological Branch of the Canadian Government it appears that the temperature at a depth of 15 in. to 2 ft., rarely goes below 15° F., in spite of air temperatures as low as 50 to 60 deg. below zero. Therefore, even though frost may penetrate to a depth of 6 or 7 ft., the temperature for most of this depth is within a few degrees of the freezing point. Any intense degree of frost is concentrated in the first foot or so of depth.

When salt dissolves in water, the freezing point is lowered. A 5 per cent salt solution will not freeze above 15° F. Salt might therefore be used to lower the freezing point of soil-water at shallow depths, below any temperature to be normally expected, or to so reduce the amount of freezing and the rate of freezing in a heaving area, as to make its movement uniform with the movement of the adjacent sections of track.

When the track movement is uniform, there is no problem. In the application of salt, the idea is to *make* soil movements uniform, by cutting down on the frost action and ice formation in the soils containing excess moisture, so that they will behave the same as the adjacent soils.

During the past three years our railways have treated some 940 spots that had been subject to recurrent and persistent frost heave, by methods based on the above theory.

Salt has been applied in several ways:

(1) Spread on top of the ballast with just enough cover to prevent it being washed off the ballast section with heavy rains. It is then worked out of sight with standard hoes or shovels.

(2) Spread in the desired thickness on the top of the ballast. The ballast section is boxed by building a lip or dyke at the bottom of the ballast section on the sub-grade approximately on the grass line. The small dyke about 9 in. is built to prevent the salt from being washed away from the ballast section. In this method the salt is not covered.

(3) Remove the ballast between the ties and place the salt at or below the bottom level of the ties. This method places the salt near the source of the heaving. For small thickness of salt, the ballast is replaced immediately, but in heavy applications it may be necessary to replace the surplus ballast left over from refilling the excavation, at a later date.

(4) The fourth method is to dig holes at 2 ft. centers, 6 in. in diameter, and varying in depth from 18 in. to 2½ ft. On the standard ballast section this places five, 6-in. holes across the track, three holes between the rails and two outside. Three inches of salt are placed in the holes and the ballast thrown in on top. The shallower holes are dug with standard tools, but a post hole digger should be used on deep holes to reduce cost.

In all methods of application, the salt must get to the moisture in the soil, and must have time to be distributed over the area before it can be expected to do any work. This means that it must be applied long enough before freeze-up to give it time to be absorbed into the soil moisture. Therefore the best time for application is in the late summer or early fall. Heaving spots treated too late in the fall give no results the first year. In

most cases, however, the salt has been effective the next season.

Run-off must be provided in applying salt, the same as when "shimming." This is done by thinning out the application towards the ends. The length of run-off is usually governed by the length of the section which has previously required "shimming." The run-off should be about half of the distance, hence one tapers off the salt gradually to a point about the middle of each previously shimmed section.

If one knew the exact amount of excess water involved in a heave spot and the temperature it would fall to, it would not be hard to calculate the exact amount of salt needed to overcome the tendency to heave. But, since the actual moisture content of the soil may change from day to day, and since the temperature follows the general trend of the weather, exact dosages of salt for treatment of any given heave site cannot very well be determined. Therefore the amount of salt required in each case must be estimated, and we have found that this can best be done by one familiar with local winter conditions, preferably the road master or section foreman.

The following rule has been developed from practical tests:

Apply salt to the hump, not in the area where shims have been installed, and apply in the same thickness as the thickest shim usually used up to a maximum of three inches of salt.

The results of the treatments to date are naturally variable. A reduction in the amount of "shimming" required subsequent to treatment has been reported in approximately 90 per cent of locations, varying from 25 per cent reduction to complete elimination,—the latter in 28 per cent of the treatments. In all cases where cost records have permitted comparison of maintenance costs for successive years, the chemical method of heave control has apparently effected substantial savings. We do not regard this, as yet, as being fully conclusive since different conditions necessarily prevail in successive winters.

When salt is first tried, the user may sometimes get the impression that it produces soft track. This can happen, particularly if too heavy an application is made in clay soils. Upon investigation of many of these situations however it has been found that the so called "soft spots" were places where the salt had done its job, but that it had been cut off too abruptly. In other words, the salt treated spot had stayed down, while the rest of the track had lifted.

The length of time over which one treatment will continue to function cannot be predicted accurately, as soil conditions naturally vary from place to place. Our records support the conclusion that two years is the minimum expectation, as many of our original successful treatments are still functioning satisfactorily.

If the thoughts of this paper were to be translated to the field of road construction, they would support the use of salt as an admixture in the sub-base courses of highways, as a straight out-and-out frost control measure, with no particular reference to stabilization. This procedure has in fact been followed in the construction of Nova Scotia highways for the past two years, where 20-40 tons of salt per mile have been mixed into the sub-bases of newly constructed highways for no other reason than frost action control. The results, we understand, have been entirely satisfactory.

SOIL SURVEY PRACTICE IN THE UNITED STATES

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SYNOPSIS

This paper is a summary of replies to a recent questionnaire submitted by all the States on their soil survey practices. At present, all but 4 States are making such surveys, although 12 of these do so on a very small scale only. Of the other States, most follow, in general, the methods and procedure outlined by the A.A.S.H.O. and the Public Roads Administration. Two States make their surveys by methods outlined by the U. S. Bureau of Chemistry and Soils, classifying soil by the pedological method, rather than by the triaxial chart or the Public Roads classification. Other States also make use of the soil maps prepared by the Bureau of Chemistry and Soils in addition to following A.A.S.H.O. procedure.

Subgrade explorations are being made by means of test pits, 1½ in. standard soil augers, 4 to 10 in. post hole auger, and mechanized drills up to 36 in. diameter. The definite trend is to use of the larger post hole augers. Some States also use power samplers for taking 2 and 4 in. diameter undisturbed samples. Some subsurface exploration is also being done with electric restivity apparatus.

The extent of sampling is variable, as in some States samples are taken only where the suitability of material is questionable. Other States take samples at regular intervals along the line. Some States take samples only for visual inspection and general comparisons, while others make detailed field or laboratory tests on the samples.

It is recommended that sufficient borings be made to provide the accurate mapping of all soil horizons.

The location and direction of underground water flow is of great importance in subgrade soil behavior. It is suggested that all borings be extended to at least 3 ft. below subgrade elevation.

Before making a soil survey, consideration should be given to the purpose for which the results are to be used; a survey for the foundation of a structure must be conducted differently than the survey for a highway location. This paper deals with the problem and general practice of making a subgrade soil survey.

The purpose of the highway subgrade soil survey is to furnish the highway engineer information from which he can more accurately design a section of road. Formerly a material which looked good was considered good, and with the very meagre information available it was practically impossible to identify those materials which would perform badly or which would perform well. Likewise, it was equally impossible to predict those areas which would fail either in subgrade, embankment, or foundation; and it was equally difficult to account for many landslides and slips, which constantly occurred during construction or shortly after the completion of a project.

We have all had experience with pavement failures, which by all the known laws should not have occurred. We have tried extensive drainage but still we have had innumerable failures. These failures have finally brought forth the conclusion that they are probably developing through subgrade soil action. This conclusion has called for a thorough investigation of subgrade soils. Since the science of soil investigation is so very new, and the technique has been developed so rapidly, it was deemed advisable to make a survey of the methods being used by the various states.

A questionnaire was sent to all states requesting information concerning methods of making the preliminary soil survey, including underground explorations and sampling, and of the use made

of the data obtained from the survey. As evidence of the interest being taken in this subject, it can be reported that all states replied to the questionnaire. This paper is a resume of the replies.

Since this paper is essentially a review of the methods now being used by the various agencies conducting soil surveys, we will quote at some length from the replies.

Alabama reports that the preliminary soil survey is run between the completion of the location survey and the drafting of the plans. Complete soil surveys are run only where unusual conditions are encountered, and the survey is then made according to A. A. S. H. O. methods. Alabama stresses the point that in the great majority of cases, a survey of the scope outlined by the Association is not necessary, so in these cases samples are taken at 500-ft. intervals only for study and for record. "Where the alignment on a new project follows an existing road, sampling is done at 500-ft. intervals to a depth of 12 in. below the proposed subgrade elevation, and from the test results on these samples, it is decided whether the existing roadway is a suitable subgrade; and if not, what will have to be done to make it so. Samples are also taken from the banks of existing cuts and from other sources of material for widening to determine its proper distribution and place in the completed work."

Alabama has a contract item, known as "Roadbed Topping," which consists of a compacted blanket of selected soils used to reinforce weak subgrades and ranges in thickness from 6 to 15 in., depending upon comparative experience in the same locality with soils of like characteristics. The preliminary soil survey here helps to determine where such topping is necessary and the required thickness.

The most common type of equipment used in making soil borings is the post hole auger of 8-in. diameter. This auger has been used successfully to a depth of 27 ft. and is found to be generally superior to augers of smaller diameter.

Alabama places a great deal of dependence in soil studies, and believes they have contributed greatly toward the success of the low cost road program in the state. Data, obtained from these surveys, are used fully in the preparation of plans and for the guidance of the resident engineer during construction.

Arizona has not as yet adopted a method of making underground explorations. In making their preliminary soil surveys in open country, test pits are dug approximately 1½ ft. deep each quarter of a mile. When cuts are encountered, test pits are dug to grade, providing the material can be readily excavated with hand methods. Wherever rock is encountered, no attempt is made to determine the extent. Sampling is done by making a cut down one face of the test pit, thus taking a composite sample of the stratum under investigation. Care is exercised to see that other strata are not included.

Arkansas makes borings with a soil auger along the centerline of the proposed location to the depth of the grade line in cut sections, during which a visual inspection is made to determine the depth of each soil layer. Sufficient borings are made to define accurately the lateral extent of each of the soil types encountered. These intervals are determined by the soils engineer making the survey. Representative samples are taken of each layer and sent to the laboratory for classification.

In California a study is made of the subgrade conditions on each project and is started while the preliminary location surveys are underway. This is done in order that the information may be available early in the design period; so that maximum economy may be obtained through the use of local materials insofar

as possible. The routine soil surveys are made by the District Materials Engineer, and a soil profile is furnished as a portion of his completed report.

Mr. O. J. Porter, Senior Physical Testing Engineer of the California Division of Highways, in his paper, entitled, "The Preparation of Subgrades,"[1] gives the method used in California in making the soil survey. He says in part:

"Although the scope of investigation is usually dictated by the nature of the terrain and other project conditions, we usually first study the geology of the region; second, make borings to expose and sample the typical materials; and, third, perform tests to determine the essential properties of the soil. A proper balance between these three methods of attacking a subgrade problem will usually produce adequate data at minimum cost.

"The most common type of exploration used in this state consists of hand borings made with a soil sampler, similar in design to the larger power-operated sampler described by T. E. Stanton in the Proceedings of the International Conference on Soil Mechanics, Volume I, June, 1936. This outfit, driven either by hand or with a light, portable gasoline hammer, is suitable for obtaining 1 in. cores from penetrable ground to depths ranging up to 50 ft. Such borings, supplemented by tests on larger samples from relatively shallow hand auger holes, generally furnished sufficient information for preparing a soil profile through light grading work on valley projects.

"Explorations of a larger size, however, are often required adequately to disclose the materials available in heavy cuts on larger grading projects, and also to investigate any other local sources of material proposed for subgrade reinforcement. In such cases, larger borings are made with a special combination drilling rig, designed and built by the department for (a) heavy duty churn drilling, (b) making rotary borings up to 36 in. in size through earth and soft sandstone and shale formations, and (c) operating the larger size of sampler outfit.

"The churn drill is used to actuate a hammer for driving the sampler outfit, and also for breaking through rock and boulders not readily penetrated with the sampler and rotary tools. The larger diameter rotary borings are made to depths of 50 to 80 ft. to investigate ground water conditions and to examine the formations in place. Large undisturbed samples may also be obtained from the portion of the ground explored with the rotary tools."

The data, obtained from the soil survey, are used for the following purposes:

1. To determine the type of surface.
2. To determine the thickness of rigid pavements.
3. To determine the thickness of base for non-rigid pavements.
4. To determine the type and quantities of stabilizer for soil stabilization projects.

Colorado makes soil surveys using the method designated by the A. A. S. H. O. with the exception that the number of samples taken on any job is fewer than that recommended by the Association. By using glass bottles to carry small samples of materials duplications are easily avoided. Some little work has been done with undisturbed sampling.

Hand augers, similar to those recommended by the A. A. S. H. O., are used. When making surveys for borrow-pits, a Buda earth drill, mounted on a 1½ ton truck and making a 14-ft. hole, 20-in. in diameter, is used. This machine enables the engineer to make a thorough investigation of any proposed borrow or gravel pit.

Connecticut reports that they have never made any attempt to analyze the sub-soils encountered, depending entirely on a general knowledge of conditions in the various parts of the state for the determination of the thickness and type of base or surface to be used. However, prior to any preparation of plans for a new project, a very thorough survey of the service record is made of the road to be improved. From this survey, location of springs, rock, and general soil conditions are observed and used in designing the new roadbed.

Delaware follows closely the procedure outlined by the A. A. S. H. O.; however, as many test holes are drilled as may

[1] *Proceedings*, Highway Research Board, Vol. 18, Part II, p. 324, 1938.

be necessary to determine the character and extent of all soils through the project. At least one hole is drilled wherever the grade changes from cut to fill, or level section, and wherever the presence of rock or underground water is thought likely. Considerable care is taken to locate sub-surface water, and a careful study of the source and best method of drainage is made.

Samples are taken of each material encountered and the routine tests are made for identification, according to the Public Roads Administration standard group classification. The suitability for various purposes of the materials encountered is determined by means of their group classification, and by knowledge of special local soil types or conditions.

The equipment used in making the soil survey consists of three sizes of the ordinary open type post hole auger of 4, 6, and 10 in. diameters with extensions for drilling to depths of 30 ft. An adjustable closed type post hole auger having one hinged side, which drills holes from 10 to 12 in. in diameter, is also included as standard equipment. It has been found that the smaller size augers are not satisfactory, since they do not remove a sufficiently large sample for testing purposes.

The methods used by Florida are practically the same as those set forth by the A. A. S. H. O., with the exception that a soil profile is not drawn. In making the underground exploration, post hole and soil augers are used. Care is taken in the exploration work to determine and locate all strata and points of change. Samples are taken of each soil type encountered. The soils engineer in charge of the investigation makes the field classification and determines what samples should be submitted to the laboratory for classification. The data collected on the soil survey are used in preparing plans and specifications for the project. The test results are shown on the plans as also are the methods of treatment of the unsatisfactory materials encountered.

In Georgia the general procedure followed is that after the centerline and approximate grade line of the new project have been established, the soil survey crew goes over the project making borings to about 4 ft. below the preliminary grades, taking samples of each stratum encountered. The borings are made with a 4-in. hand prospector's auger; the frequency of such borings depends upon the uniformity of the soil. Borings are taken often enough to establish the length and depth of all strata encountered. Only about two samples of each soil stratum are taken for each ten miles of roadway. In addition to the borings, all existing roadway cuts and open wells in the vicinity of the project are noted and studied. The information thus noted is used to establish the finished grade, to determine subgrade construction methods and thickness.

In Idaho the procedure is briefly as follows: (1) A brief reconnaissance is made over the proposed route taking note particularly of geographical features, drainage, nature of soil, outcroppings and extent of such, and in case of a realignment, particular attention is devoted to sections of failure on the old road and if possible cause of failure is noted. (2) Borings and test holes are then made in order to determine the extent, both horizontal and vertical, of all soil strata. Samples of each stratum encountered are taken to the laboratory for testing. In order to avoid excess sampling small samples of each soil type encountered are carried in small air-tight bottles. These samples are first classified as to texture, color, structure, and cementation. This enables a rapid comparison with samples already taken as each new stratum is encountered. This general procedure is followed until all soil types

are logged. This underground exploration is made wherever possible with 1½-in., A. S. T. M. type soil augers. However, in gravelly soils test pits are dug when the auger cannot be made to penetrate. An attempt is always made to secure information to a depth of at least 2 ft. below the grade line. (3) The data thus obtained are used in designing for drainage, and for total depth of base and surfacing materials.

Iowa makes soil surveys only on those projects on which, after a preliminary field survey, such investigations are deemed warranted. Borings are made with 4 and 6-in. augers, and are carried to a mean depth of at least 2 ft. below grade line. Auger holes are spaced 100 ft. apart where there is uniformity in the soil layers and 25 to 50 ft. apart where the soil layers are more variable. A sample is taken from each soil layer for laboratory investigation. Core samples are also taken. These core samples are carefully sealed with parafine and shipped to the laboratory for determination of undisturbed densities and moisture content.

Field notes include information concerning location and direction of flow of underground water, extent of road failures, and a field sketch of the soil layers in profile and cross-section. This information is used to supplement the laboratory data. The information from these two sources is then used in working up the completed soil profile and for making recommendations for the treatment of the materials encountered on the project. These recommendations, estimates of quantities, and location of available treatment materials are included in the final plans of the project.

Indiana reports that soil surveys are conducted on all proposed projects as soon as plan layouts and proposed grade lines are available. Although a complete soil survey is conducted throughout a project, a soil profile is drawn only in those areas where definite and detailed soil stratification is encountered. In other areas a detailed log of the borings is tabulated. Borings are made with a standard 1½-in. soil auger to a point at least 4 ft. below the finished grade and often enough to record completely the depth and extent of all soil strata encountered. A representative soil sample of each stratum is taken for laboratory analysis. In making the field investigation, whenever any unusual moisture conditions are encountered the porosity, permeability and drainage characteristics of the soils are studied and reported. Heavy fill foundations are examined for stability.

Illinois conducts soil surveys after the methods of the A. A. S. H. O. and A. S. T. M., making a soil profile for use in design and construction. In the realignment of old roads, care and study are given to locations of points of failure, to the topography, surface drainage areas, surface geology, source and direction of flow of underground water and other pertinent data. Borings are made with soil augers at 50-ft. intervals unless changes in soil types require closer borings. All investigations are carried to at least 3 ft. below grade line. Borrow areas are also fully investigated. Illinois stresses the point that the soils engineer in charge of a survey party should have a basic knowledge of soil mechanics, drainage, and highway construction.

Kentucky has done little work on complete soil profiles. Most of the work to date has been confined to that necessary to the construction of stabilized base courses. Underground exploration has been confined to test pits and some work with soil augers.

Kansas conducts soil surveys for ordinary earthwork construction, for the determination of the surface type, for subgrade treatment or base course construction, for foundations, for large fills or ordinary fills on side hill locations, for

unstable soils, and for underground water problems. Those surveys are generally conducted according to the A. A. S. H. O. methods; however, where the soil is of a sufficient depth and extends over a relatively large area, the soil survey methods of the U. S. Bureau of Chemistry and Soils are used.

All work on the soil survey is done by hand methods using hand equipment. Materials too hard to be penetrated by pits or by augering are in most instances classified as rock and are investigated either where they are exposed or by means of a core drill. The ordinary 4-in. orchard type auger has been found to be most desirable for making borings. Test pits are used through gravel and in rocky locations where the depth of rock is not excessive. Borings and test pits are made sufficiently close to determine the extent of the various soil types and are generally carried to a depth of two or three feet below grade line.

Rock and shale are located, and the strike, dip, and extent are determined by a geologist working independently of the regular soil survey party. Although the regular soil survey party handles all drainage problems met in the soil formations, the geologist makes a special study of all drainage features connected with rock formations.

Density tests are made on representative soils in cut sections and moisture tests are made on the deeper soils in proposed cuts, channel changes, and borrow pits where the soil moisture is not subjected to change.

A general reconnaissance study is made of roads in the vicinity, which are located on similar soils, to determine their servicability as subgrade soils and to determine the needs for the particular type of construction planned for the project.

The soils encountered in the survey are identified by visual inspection and by approximate field tests, such as feel, bite, etc., and all distinct soil types are sampled and sent to the laboratory for the routine tests. Compaction tests are made on all subgrade soil types which are encountered on a project.

During the soil survey a few preliminary borings are made to determine by visual inspection the nature of the underlying foundation soils for the larger fills. When saturated soils are encountered, a special survey is conducted. On this special survey, undisturbed samples are taken and are sent to the laboratory for testing.

Louisiana makes the subgrade soil survey as soon as possible after the location survey and preliminary plans are completed. Borings are made with either a 2¼-in. worm type or 4-in. post hole auger and in cut sections sufficient borings are made to map completely all soil layers to a depth of three to five feet below the proposed grade line. For fill sections borings are made to a depth necessary to establish a good foundation. Where poor foundation material is found, sufficient borings are made to determine its limits. Undisturbed samples are taken with a motorized drill rig of a type known as a mobile seismograph drill which is mounted on a 1½-ton truck chassis. Samples of each soil are sent to the laboratory for tests. These test results, together with field data, are used to plot a soil profile which is used for guidance in completing the design and during construction. Recommendations are made for the proper method method of handling of the materials encountered during construction.

Maine reports that soil surveys of subgrade conditions have become a general practice in their highway department during the past five years. Borings are made with soil augers of the spiral and post hole type and are made at 50-ft. intervals. If during the plotting of the soil survey in the field, a soil layer appears or disappears, intermediate borings

are made to determine the extent of the layer. Ground water is carefully located and the direction of flow ascertained. Samples are taken from each bore hole and at least in 2-ft. intervals of depth and oftener if there is any visible change in soil type. All samples taken are sent to the laboratory for routine testing. In the laboratory any duplicate samples, of which there are generally many, can be discarded if found unnecessary. Some undisturbed samples are taken with a sampler; however, 95 percent of all samples are taken in the disturbed condition. The field men identify the soil through its visual characteristics into the various types as the survey progresses, and plot a field soil profile. When the laboratory tests results are obtained, a completed soil profile is made and is submitted with a written report of the survey for analysis and design.

In Maryland in the early stages of the soil work, an attempt was made to profile accurately all projects, taking into account every slight change in soil characteristics. Now such elaborate procedure has been eliminated by the introduction of "horse sense and practical methods." Work is now confined to locating strata of undesirable soils in cuts and taking care to see that such soils are mixed with other soils, so that the characteristics will be improved. This soil survey is conducted by the methods outlined by the Public Roads Administration and the A. A. S. H. O. In making investigations in deep cuts a 4 by 4-ft. hole is dug to a depth of 4 ft.; after which a 12-in. auger is used to a depth usually of about 15 ft. Investigations are continued below that point, using smaller augers to a depth of about 3 ft. below the proposed grade. Intensive study is made of swampy areas and for the location of ground water level and direction of flow.

Massachusetts to date has not attempted a complete soil profile, but rather has limited its investigation to the extent of peat, clay, and rock formations as a part of the preliminary study of a highway project. The investigation is limited to shallow borings with soil augers, and to shallow test pits to define the limits of shallow lying bed rock. Some work has been done with a Gish-Rooney resistivity apparatus and also with the seismograph.

Michigan has probably made the most radical departure from the accepted methods of making soil surveys. In 1925 soil surveys were started for research purposes using the system as outlined by the U. S. Bureau of Chemistry and Soils. In 1927 the first survey was made for design purposes by Dr. C. E. Kellogg, now Principal Soil Scientist, Bureau of Chemistry and Soils. Not until after 1929, however, did mapping for design purposes become a routine matter.

Mr. O. L. Stokstad, Soils Engineer of the Michigan Highway Department, in his article appearing in the 1938 *Proceedings* of the Montana Bituminous Conference, points out that the main purpose of soil classification is to provide soil information without necessitating continuous sampling and testing:

> "The important need for classification no longer exists insofar as a particular problem is concerned if its solution is provided by laboratory test results. For this reason the usefulness of the eight groups (A-1 to A-8), as developed by the Board of Public Roads, is definitely limited. This grouping emphasizes the laboratory approach to the field of soil engineering.
>
> "It does seem necessary, however, to point out the need for a more comprehensive method of classifying soil in the field approach to the soil engineering problems.
>
> "The previous mentioned eight groups do not lend themselves to the recording of detailed field information without a large amount of field work and voluminous special notes."

In making a soil survey in Michigan the great soil groups are divided into soil series having similar profiles, drainage, topography, and parent material; and these series are further classified into loam, silts, and clays. It has been found

that the soil type may be used as a basis for the classification of physical and chemical data. "When used in this manner it provides a very useful tool in the study of soils for highway purposes. The Michigan State Highway Department in adopting this method of studying soils was immediately able to take advantage of the soil survey information available in the form of published county soil survey reports. In addition, it was also possible to profit by contact with the personnel of the Soils Department of Michigan State College."

Michigan has prepared a chart in which a few of the important design questions are answered in tabulated form. This makes soil information much more available to the average designer and has materially increased the utility of the soil survey. Michigan feels that the soil survey is the basis of all design, and as such, the information should be readily assembled in making the survey and the final results should be in as simple a form as possible for use in design and construction.

Mississippi makes subgrade soil surveys similar to the methods recommended by the A. A. S. H. O. Borings are made and samples taken with a 3-in. pod auger. When dense clay or hardpan is encountered, a chisel bit auger is used, and in gravel a spiral auger has proved satisfactory. These are all 3-in. augers and use $\frac{3}{4}$-in. pipe for the extensions. Borings are made at every station, unless a different type of material is encountered. Then intermediate borings are made until all layers are fully and completely mapped. A soil profile is plotted from the information collected in the subgrade soil survey. Information, concerning drainage problems and borrow pit areas, are also made available as a part of the information received from the survey. This information is all used during the design period and acts as a guide for the engineer during construction.

Minnesota uses, in general, the procedure outlined by the A. A. S. H. O. although the plotting of the data into a soil profile is not always carried out. The data obtained are summarized by the field men on special sheets thus presenting the data in tabular form. Sampling is very limited. Samples are taken only for check on the field identification, or where unusual border-line materials are found. Generally no samples are taken if the soils are of kinds whose characteristics have become well known from observation and experience. Undisturbed samples have not been taken. By means of the soil survey Minnesota establishes the relative merits of the different horizons of the soil profile; locates the positions of layers or pockets of undesirable soil materials, and areas and direction of flow of water-bearing strata.

Missouri uses the pedological system, as used by the U. S. Bureau of Chemistry and Soils, in making soil surveys for highway purposes. Missouri takes an attitude similar to that of Michigan, in that, of the three best known methods of classifying soils, the pedological system seems best to suit their needs. The other two methods seem to be limited in their scope because the classification by texture, as in the triaxial chart takes into account no other soil properties and the classification as developed by the Public Roads Administration is more an index of the behavior of disturbed soils and so does not take into account field conditions. The pedological system is essentially a reflection of drainage, topography, parent material, weathering agencies, as well as textural class. Missouri, however, does require that samples of each horizon of each soil type be taken and submitted to the testing laboratory for analysis. Field density tests are also made of each horizon in order that the theoretical soil shrinkage can be calculated for each horizon. Also in the light of past experience and test results, it is

sometimes possible to recommend a treatment or placement of some particular soil materials in order that they may render better service in embankment or subgrade. Normal drainage is studied and any combination of soil or stone and soil materials, which would seem to create a critical situation as related to either normal or diverted drainage, is given special and careful consideration.

Montana follows closely the procedure recommended by the A. A. S. H. O. An auger similar to that recommended by the A. A. S. H. O. is used and borings are taken often enough completely to record the various soil layers. Borings are made under low fills to a depth of 3 ft. and under high fills to a depth of 6 ft. In cuts, borings are taken to a depth of 6 ft. and nearby exposed cuts are examined and deductions made from these. Samples are taken for laboratory analysis of each soil layer encountered.

Nebraska makes preliminary soil surveys in a manner similar to the method outlined by the A. A. S. H. O. Underground explorations are made with the hand auger or by digging test pits. Sampling is done with the augers and the frequency of taking samples is determined from visual classification made by the engineer during the survey. Nebraska at the present time is attempting to correlate data obtained from their soil surveys with the pedological data that have been assembled for the state by the U. S. Bureau of Chemistry and Soils and the University of Nebraska. This work has been carried on by the Bureau in Nebraska for 25 years and as a result, soil maps exist for practically every county in the state. Nebraska hopes to work out a procedure similar to that used by Michigan and Missouri.

Nevada uses a system of making soil surveys similar to that of the A. A. S. H. O.; however, they report a little different method of testing and sampling deep cuts. In deep cuts a series of holes are sunk from below grade line over the top of the ridge to be cut, attempting to sink each hole down to the stratum encountered at the top of the preceding hole. These holes are dug at intervals the entire length of the proposed cut. Thus they are enabled to map fairly accurately the profile of any cut. Samples are taken of each material and submitted to the laboratory for analysis.

In New Hampshire the preliminary soil survey is started in the winter and spring when it is possible to make a survey of all frost heaves and uneven sections. This enables special attention to be given to those areas which otherwise might be missed. Borings are made at 100-ft. intervals to a maximum depth of frost penetration. Wherever ledge is suspected, sufficient borings are made to show the profile of the ledge to help in estimating quantities and to eliminate, during design and construction, any pockets which might be formed by the ledge so that underground water will not be impounded. All swamp and muck areas are explored with a regular soil auger and a Michigan muck and marl sampler.

After the soil survey is made the information gathered, together with the laboratory analysis, is transferred to the road profile and colored according to type, so that it may be easily seen what materials will be found in any cut. The information thus gathered is used both in the design and in the construction.

New Jersey reports that they do not make subgrade soil surveys. They do, however, dig test pits to examine the ground for general soil types and location of rock. In areas of soft ground rod soundings are taken to determine the depth and consistency of such material.

New Mexico has only just started making soil surveys and has not as yet established a standard of procedure. Samples are taken for the most part only when visual and manual inspection of the ma-

terial gives rise to any reasonable doubt as to the suitability of the material. Data gathered are used as a guide to judgment in making recommendation for use in construction on for remedial measures.

New York reports that to date only a few soil surveys have been made. The methods used are similar to those outlined by the Public Roads Administration and the A. A. S. H. O. Samples are taken either from test holes or from auger borings and are tested in the laboratory for classification according to the A. A. S. H. O. methods. Several types of diggers and augers are used in obtaining samples. They have used successfully a Pincer type post hole digger, a 4-in. and a 6-in. diameter post hole auger, and a hinged spoon. The information obtained from the survey is used to determine methods of stabilization of materials placed in embankments and found in unstable cut sections.

North Carolina, except in special cases, does not make detailed subgrade surveys. They do, however, make visual examination of subgrade soils prior to letting a contract and then make a thorough examination during construction. Sometimes when seepage is encountered a soil profile is made from data obtained from borings and the condition is adjusted either by grade change or by sub-drainage. Underground explorations are made with soil augers and samples of the different strata are sent to the laboratory for classification according to the A. A. S. H. O. requirements.

In North Dakota preliminary inspection is made of the project to determine the main characteristics of the soil profile. From cuts and borings the different soil layers are determined and, depending upon the type of soil encountered, about three samples per mile are taken and sent to the laboratory for analysis and classification. A continuous soil profile is plotted showing all soil strata and laboratory test results. Field densities are obtained for use in predicting shrinkage factors. The soil profile and all other information is used both during the design and during construction of the project.

Ohio began routine soil testing in 1934. The soil profile includes the mapping and presentation of data from a complete investigation of all soil and geological stratifications which may be encountered throughout a project and which may influence the design, construction, and maintenance of the section. The information for the subgrade soil profile is obtained by borings, test pits, and the examination of exposed cuts. Borings are made with 4 and 6-in. post hole augers and with 1½-in. soil augers and are made at such intervals as are necessary for complete mapping of all subgrade strata. These intervals may be as close as 25 ft. or as far apart as 500 ft. depending upon soil conditions. A small sample of each new layer as it is encountered is taken for comparative purposes. This reduces sampling to a minimum and still insures a complete and accurate profile. Field density measurements are made for the purpose of determining a shrinkage factor.

Ohio has experienced considerable difficulty with landslides. Investigation indicates that causes of slides can be classified under five general headings, namely, unconsolidated soil, unsuitable material, settlement, side hill fills, and slipping plane.

In order fully to investigate the causes of a slide it is necessary to go carefully into records which might have a bearing. Ohio has found that the following information is important:

(1) Time of year when slide was first noted.
(2) Weather conditions.
(3) Past efforts for stabilizing the slide.

(4) Records of all slides in immediate vicinity.
(5) The presence of coal mines.
(6) Maintenance records on slide area.
(7) A careful geological log of the area.

The first step used in sampling a landslide is visual inspection. Samples should be taken from areas near the top, part way down on the slope, and near the toe. Samples are taken with a 6-in. post hole auger or by core borings. Undisturbed samples are taken near the toe. Density measurements are taken in all three areas. Samples are tested in the laboratory to determine the moisture content, mechanical analysis and other physical tests to classify the soils. On the undisturbed soil samples consolidation and shear tests are run. From all the information gathered from both the field investigation and the laboratory tests, it is generally possible to determine cause of slippage.

In Oklahoma the general practice has been to investigate the subgrades on those proposed grading jobs on which it is expected to place rigid-type surfaces, or on completed grades on which such pavements are to be placed. Only in very special cases are subgrades sampled where flexible type construction is to be used. In proposed new construction samples are taken in excavation of each stratum or change in soil type to 2 ft. below grade line. Such additional soundings in the same excavation are taken as seems necessary depending on the length of cut and the general conditions in the areas as to uniformity of the soil horizon. The field notes and the tests are studied in connection with the plans for the project and recommendations for correction of the soil as seen necessary are included on the finished soil survey report.

Soil surveys in Oregon are made by the location party during the location survey. Underground exploration is made by means of a soil auger or from open pits or shafts. Sampling is ordinarily done by the location crew using the auger or from the test pits. Samples are taken from each layer and are sent to the laboratory for analysis. Data obtained from the survey are used to determine the construction procedure and are used by the locating engineer in arranging the distribution of material. Particular care is taken in locating springs and underground flows as an aid in providing proper drainage. Soil surveys have only been under way in Oregon for about one year and the A. A. S. H. O. and Public Roads Administration classifications and methods are now generally used. Prior work was limited to the practical knowledge which the various engineers had obtained from the use of soils.

Texas conducts a complete soil survey much after the pattern set forth by the A. A. S. H. O. The organization of each of the 25 districts of the state includes two or more engineers who have had the advantage of an intensive training course in both the fundamentals of soil mechanics and the application of soil mechanics to design of roadway facilities. At the present time subgrade surveys are conducted under the direct supervision of these soil engineers and generally follow in convenient sequence the work of the location party. Methods of making soil surveys vary according to conditions. Exploration is done using soil augers, bars, picks, shovels, and post hole augers, and is carried to a depth of at least 3 ft. below the proposed grade. Excavations are carried 3 ft. below ground line where embankments are to be placed. The frequency of borings is determined by the extent of various soil strata and sufficient borings are made to define all strata both horizontally and vertically.

Samples are taken for each stratum encountered during the boring operations. As samples are obtained they are first

grouped in accordance with the apparent visual characteristics, such as, color, texture, grading, and structure. This grouping is only for information and does not establish any similarity of quality of the soils. When this preliminary grouping has been completed, a sample of each of the materials is prepared for the linear shrinkage test which is made in the field. This is an unusual procedure and enables Texas to further classify soils in the field into more definite groups. Those samples within the preliminary grouping having equal linear shrinkage may be considered as soils having uniform characteristics and to have come from the same or similar formations. Using this basis of grouping, it is possible to take a few samples for submission to the laboratory for determination of the soil constants.

Information from both the field survey and from the laboratory test results is used for the development of a continuous soil profile for the project in question. Soils are grouped for indication on the soil profile by a process of selection and combination. First, the soils are visually inspected and linear shrinkage tests made. Those samples having similar visual characteristics and linear shrinkage are of the same character and are grouped together and a sample taken from each group for making routine laboratory tests. Second, the soils are finally grouped for indication on the soil profile by range of plasticity index ranging from 0 to 4, from 4 to 15, from 15 to 25, and above 25. From this soil profile the distribution of excavated material is determined. It further permits analysis of other features, such as the requirement for underdrains, the amount and type of consolidation required, and the moisture content necessary for desired density.

Utah conducts a complete soil survey similar to those outlined by the A. A. S. H. O. and the Public Roads Administration. Borings are made using post hole augers and standard 1½-in. soil augers, and if further exploration is necessary, pits are dug. The frequency of the borings depends on the strata encountered. Enough borings are taken to insure that all strata are fully mapped. Such explorations are carried to 5 ft. below the proposed grade line.

Samples are taken of each strata and submitted to the laboratory for complete analysis. A good deal of dependence is placed on the results obtained in testing both disturbed and undisturbed samples in the stabilometer, from a field bearing determination and from a study of general geological characteristics of the area. Care is taken to locate ledges of rock, water bearing strata, and any swampy or boggy areas. Field density determinations are also run using the standard Proctor method in order that shrinkage factors may be calculated for the materials met. Field moisture tests are also made to assist the design department in the determination of the amount of water necessary to obtain maximum density during the construction period. A complete soil profile is made by the soil engineer from the results obtained during the field investigation and laboratory tests. This information is used to assist during the design period and during the construction of the project.

Soil surveys in Vermont are conducted paying particular attention to land contours, drainage, vegetation, stoniness of soil, presence of boulders or ledge, exposed cuts, etc.

Special attention is given to swamp areas, filled in or made land, and water table levels. Underground exploration is made by the use of the post hole diggers and augers. The augers being from 1½ to 2-in. diameter. Bars for soundings are also used. Ordinarily field observations of soil removed from the borings or test pits are deemed sufficient analysis, but occasionally samples are taken for grading analysis and determination of stabil-

ity in the laboratory. The subsoil data, when plotted on cross-sections and profiles gives the design engineer information concerning swampy and boggy conditions, underdrain requirements, rock quantities, and other similar information.

While Vermont does not conduct a complete soil survey according to the A. A. S. H. O. requirements, they do place a good deal of dependence upon their method, as outlined.

In Wisconsin soil surveys are conducted only in sections of the state where trouble with subgrade disturbances have been met. Equipment used consists of soil augers, post hole diggers, shovels, peat samplers, etc. On the preliminary soil survey the auger is used to a depth of 6 ft., save where marshes or peat swamps are encountered, in which case, the exploration is carried to a point deep enough to obtain solid footing. When and where the proposed grade line involves cuts greater than 6 ft., the procedure is to discontinue the preliminary soil survey at that point and subsequently continue at the time the rough grading work through such area is approximately complete. Any remedial measures necessary are decided upon at that time and executed while grading equipment is in the vicinity. Generally the soil survey starts with spacing auger holes at 25 ft. intervals using a serpentine design within the roadway limits. When any trouble potentialities are encountered the spacing of the borings is varied to locate the limits of the troublesome areas. Wisconsin has also used quite extensively an electrical resistance apparatus in making underground explorations. Marshes and swamps are explored with the use of sounding rods and peat samplers, and water table levels are carefully located.

The number of samples taken is very limited since much dependence is placed upon the present knowledge which the field personnel has developed about the major types of Wisconsin soils. The average number of samples taken does not exceed one to a mile. Samples are obtained with the soil auger or post hole digger and are transmitted to the laboratory for complete analysis using the standard P. R. A. classification. To date no undisturbed samples have been taken.

A complete soil profile is made using the data obtained from the field survey and laboratory tests. The results obtained from the soil survey data are used both in design and construction.

Wyoming has not yet entered fully into the field of underground exploration. They have recently purchased an apparatus similar to that used in California for the drilling and sampling of subgrade soils and with this equipment they expect to make complete soil surveys. Until the present time, such exploration work has been confined to shallow depths and there has been no attempt to follow the procedure outlined by the A. A. S. H. O. Some sampling was done with the methods of sample taking and the number, being left entirely to the discretion of the field or project engineers.

Tennessee, Virginia, West Virginia, and Washington have only conducted small scale surveys, such work having been done under the methods prescribed by the A. A. S. H. O. The explorations have generally been limited to areas where it was apparent from visual observation that difficulties might arise in construction from unsatisfactory conditions. Data obtained from such surveys have been used to determine where settlement is apt to occur, supporting power of soils under fills, and frost heave danger.

Pennsylvania, South Carolina, South Dakota, Puerto Rico, and Rhode Island have not yet conducted subgrade soil surveys; however, such operations are planned for the future. Rhode Island feels that because of the large, state-wide deposits of gravels of all gradations which are available at a cheap rate widespread soil investigations are unneces-

sary. An annual survey is made in late winter and early spring to note and remedy any failures.

Hawaii has only recently started subgrade soil studies. Each project is studied and tested in accordance with the terrain. The frequency of sampling and the methods used in taking samples are left to the discretion of the resident engineer. A special test machine for determining the bearing power of soils has been developed, which has proved very useful in the designing of subgrades and surfaces.

As a result of this study it is evident that today the generally accepted procedure in making subgrade soil surveys is similar to that outlined by the A. A. S. H. O. and the Public Roads Administration. The reports indicate that while all States using this method do not now make complete soil surveys and complete soil profiles it has generally been expressed that such work will shortly be done.

Subgrade explorations are being made by means of 1½-in. standard soil augers, 4, 6, 8, and 10-in. post hole augers; this equipment being supplemented with bars, pincer type diggers, spoons, picks, and shovels. A great many of the states continue to rely upon test pits as the only satisfactory means of subsurface exploration. However, the trend is quite definitely in the direction of using the larger type post hole augers.

The use of these augers means a considerable saving in time and effort over that required for test hole digging. These larger augers also have the advantage over the smaller size auger inasmuch as a large enough sample can be removed for comparison and testing purposes.

A number of the states report the use of portable, mechanized earth drills, boring test holes up to 36-in. diameter. While these rigs are costly they do make possible an accurate record of the underlying soil strata. However, such large equipment is generally considered unnecessary as adequate and accurate work can be done with much smaller equipment. California has for years successfully operated a power sampler taking 2 and 4-in. diameter undisturbed samples. Many other states report equal success. Borings have been made to depths ranging between 60 and 300 ft. at an average cost of approximately $1.00 per foot including rental of equipment and all operating costs. Similar sampling with other methods usually costs two to four times this amount.

Some subsurface exploration work has also been done using resistivity and earth resistant apparatus. These methods are still quite definitely in the experimental stage and hence have not as yet been adopted or used very extensively.

Wisconsin reports, "In the last two years we have been using not infrequently the electrical resistance apparatus. Our efforts with this equipment have been rewarded in most cases with a satisfactory degree of accuracy and in some instances we have been 'let down' with some information not too good." Massachusetts reports in the same general trend, however, emphasizing that the work was done only experimentally.

States generally agree that sufficient borings or test pits should be made adequately to map all strata encountered and should be deep enough to be 3 to 5 ft. below the grade line. This generally supplies enough information for the proper design of the roadway facilities. Many states report that the exploration is carried down below the grade line until a water bearing stratum is encountered or until a relatively impervious layer is met. This provides adequate information for the complete design of all underdrains.

Determination of the number of samples to be taken varies considerably in the various States. Several take samples from all test holes for submission to the laboratory while others go to the other extreme and take only samples of material of which there might be a ques-

tion as to its suitability. Most States follow the center path, that of allowing the man in charge of the soil survey to determine what samples should be tested, but requiring that a sample from each stratum be submitted to the laboratory for complete analysis.

In order that sampling might be reduced to a minimum, the various States have devised methods of making field comparisons of the strata. The general procedure is to take a small sample in a bottle of stratum for comparison. Thus samples which have similar visual characteristics, such as, texture, color, grading, and structure can be classified in the field. Texas uses the linear shrinkage test as a further means of field classification. It is well, however, to note that no one can definitely ascertain the quality of soils except by making physical tests. Soils widely dissimilar in character may appear to be identical and similarily, soils widely dissimilar in appearance may actually have the same characteristics. It is necessary, therefore, that visual inspection be augmented by physical tests for grouping determination.

To make the field survey as comprehensive as possible, field bearing tests and field density determination tests have been devised and used very successfully. California, Utah, and Hawaii have all developed bearing value tests which throw considerable light on the field behavior of soils. These tests have been devised to answer two of the most important subgrade questions affecting pavement service, namely, the amount of resistance to displacement under moist to wet conditions and the volume increase and uplift resulting from absorption of moisture, subsequent to construction. The California method of making the test as reported by O. J. Porter is as follows:

"Tests are made by wetting the material to optimum moisture content and then consolidating the sample in a 6 in. cylindrical mold under a load of 2000 lbs. per sq. in. The sample contained in the mold is then tested for bearing value by penetrating the center of the compacted specimen at the rate of 0.05 in. per minute with a piston, 3 square inches in area. A dial fastened to the apparatus accurately measures the penetration of the piston. The loads for each increment of 0.1 in. penetration to a total of 0.5 in. are recorded. Following this operation the specimen is loosened, reconsolidated, and then placed in a tank of water, after first noting the height. During the soaking period the sample is confined within the mold by a porous disc and a 10 lb. weight which represents the surcharge effect upon the subgrade of a 4- to 5-in. thickness of pavement. After the specimen has soaked for four days the swell is recorded and the bearing value again determined for the compacted and soaked condition.

"The bearing value results, for both conditions of the specimen, are reported in pounds per square inch and also in the percentage of the loads required to obtain like penetrations of a standard sample of crushed rock surfacing material similarly tested. The expansion of the specimen during the test is recorded in percentage of the volume of the compacted sample before soaking."

Hawaii uses a slightly different method. The soil sample free of rocks and stones and soil particles larger than $\frac{3}{8}$ in. is introduced alternately with water into the test pan at a rate which will insure saturation. The saturated sample is compacted under a static load, drained, released from the compaction load, surcharged with a load equal to the pavement weight and tested. The load is applied by a handwheel against a calibrated coil spring. Penetration of the piston is measured by a scale on the penetration rod. The load penetration curve is taken to indicate the comparative stability of the soil sample.

Utah has developed a bearing apparatus for making field determinations. The apparatus also can be readily adapted to use in the laboratory on either undisturbed or disturbed samples. With this machine accurate field determinations can be readily made and its use has solved some difficult bearing problems.

Many states report using field density measurements for the determination of shrinkage factors. These measure-

ments are made following the methods as outlined by R. R. Proctor for the determination of the density of compacted soils. In performing the test, the loose surface soil is removed and a hole of about 8-in. diameter and 1 ft. in depth is dug, and the removed soil is carefully weighed. The volume of the hole is determined by filling it with a fine dry sand of known density. The weight of sand used determines the volume of the hole. The moisture content of the removed soil is determined by oven drying a small sample. From these data the dry density of the soil can be calculated. The shrinkage factor can be determined by comparing the dry density of the material in place with the compacted dry density as obtained by the standard laboratory method of compaction of soil.

After the field survey has been run and the laboratory test results obtained a complete soil profile should be made. Probably the best procedure in making a soil profile is to make a rough profile in the field during the time the boring and field testing are in operation. By this method all strata can be completely and accurately mapped.

The construction of the soil profile is essentially a drawing process, requiring only superficial knowledge of soils in its preparation. It should, however, be carefully checked against the field and laboratory data by the soils engineer before it is inked or utilized in design. The ground line should be indicated by a heavy solid line and the limits of each stratum by dashed lines. The characteristics of the several strata should be indicated on the drawing so that all information can be readily available. The various strata can be shown by some conventional system of symbols, by cross-hatching, by coloring, or by any other method.

There is one very interesting trend in the method of making soil surveys for highway purposes and that is the method used by Michigan and Missouri. They are using the methods of survey as outlined by the U. S. Bureau of Chemistry and Soils. Kansas and Nebraska have also experimented using this method wherever the information is available. These states take the position that inasmuch as many years of work has been done by the Bureau in making soil surveys, why should not this information be utilized by the highway departments.

"The Michigan State Highway Department has found that the pedological scheme of soil classification provides an excellent means of studying soils for general highway purposes and that it serves as a foundation for all other soil studies in the department. The soil type, in addition to serving as a unit in mapping, also serves as a unit to which laboratory test results, road behavior observations, construction problems, and maintenance experiences are related. In other words, the soil type serves as a means for classifying and coordinating the accumulated soil experience of the entire highway department personnel.

"Until quite recently the common impression has been that the highway engineer is interested only in those soil properties upon which the serviceability of the pavement depends. It is now quite generally agreed that the highway engineer is interested in the entire highway right of way. This wider interest requires consideration of such subjects as landscaping, erosion control, shoulder stabilization, and to some extent, land use. The intelligent and efficient handling of these subjects, especially by the design division, requires a method for classifying the soil which reflects a large amount of detailed information about each individual soil."

Michigan and Missouri feel that the pedological system of making soil surveys fulfills this requirement.

SUMMARY

To summarize this subject certain definite conclusions may be drawn from the replies received from the various States.

(1) Most states find that at the present time a soil survey conducted by the method outlined by the A. A. S. H. O. adequately fills their needs.

(2) Adjustments in the methods as outlined by the A. A. S. H. O. must be made for local conditions. No set rule can be successfully followed in all cases.

(3) Sufficient borings should be made to map completely all soil horizons. The interval between borings may be as little as 25 ft. in areas of great stratification, or as far apart as 1000 ft. over areas of very uniform soil conditions.

(4) The explorations should be carried to a depth of at least 3 ft. below subgrade elevation.

(5) Areas of ledge or rock should be explored fully to determine their extent. This can best be done using a power drill of a small diameter.

(6) Samples should be taken of each soil type encountered on the project under survey.

(7) Disturbed samples can best be taken, under normal conditions, using post hole augers of 4 or 6-in. diameter.

(8) Undisturbed samples should be taken of all questionable soils. This can best be done using one of the sampling devices now being used by the various states, for instance, the Porter sampler of California.

(9) All cuts and wells in the vicinity of a project should be thoroughly studied.

(10) Care should be taken to map completely all underground water areas which might influence the subgrade soil behavior. Direction of flow should also be ascertained.

(11) Complete soil profiles should be drawn, first in the field, and finally in the drafting room.

(12) The final soil profile should contain complete field and laboratory data.

(13) The final report of the soils engineer should include, in addition to the soil profile, recommendation for the proper use of all materials, so that these materials may render more efficient service in the subgrade.

(14) The methods of conducting soil surveys as outlined by the U. S. Bureau of Chemistry and Soils certainly should be studied by states where such surveys have been made with the thought of correlating this information for use in highway design.

(15) The purpose for making the highway subgrade soil survey is in order that recommendations may be made concerning the following points:

a. Necessary revisions in the proposed grade to secure a stable and satisfactory subgrade

b. Any necessary shifting of the horizontal alignment to prevent slides or secure an improved subgrade material

c. The design and location of special ditches and subsurface drains

d. Any necessary revisions of the standard fill slopes or cut slopes

e. Any necessary subgrade treatment and the type required

f. Selection of fill material

g. Determination of peat removal and rock quantities

h. Suitability of foundations for heavy fills

i. To determine the type and quantities of stabilizer material to be used on soil stabilization projects.

FIELD TESTS ON SHEARING RESISTANCE

By Fred Burggraf

Materials and Research Engineer, Calcium Chloride Association

SYNOPSIS

At the meeting of the Highway Research Board last year a field shear testing apparatus was described and some typical data obtained with it were presented (*Proceedings*, Highway Research Board, Vol. 18, Part II, pp. 249 and 426).

This report contains additional data on completed projects which have varying degrees of service and analyzes the results. An important quantitative confirmation is that the quality of the subgrade determines the behavior of calcium chloride stabilized roads. In order to get significant results it is necessary to test the various components which make up the road structure. It was also found that the stability strength obtained by this testing device on cohesive soils is related to the bearing capacity, the bearing capacity being approximately three times the stability strength. Data are also given that show the stabilities of the frictional type subgrades are greatly increased by the surcharge effect of the overlying layers, while the strength of the cohesive type subgrades are only slightly improved.

Attention is called to the additional information needed before a tentative design method can be formulated for stabilized road structures.

DISCUSSION OF DATA

Tests Reveal That Quality of Subgrade Determines the Service Behavior of Stabilized Roads.

The general plan for the field testing was to make stability tests on adjacent

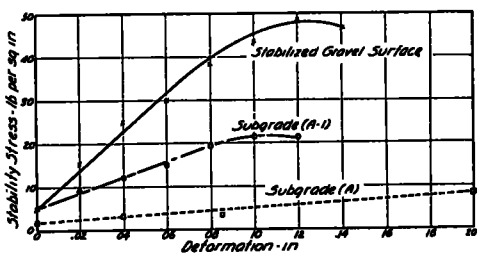


Figure 1. Project I-2
Condition: (A) Badly failed, (A-1) Good

failed and unfailed areas by testing individually the various components which made up the road structure. That is, on a road made up of a stabilized mat on a clay subgrade, tests were made on each of the components. By making these tests on both failed and unfailed areas which were adjacent, a quantitative set of results was obtained which revealed the weakest component or the one causing the failure. Such a set of test results is shown graphically in Figure 1. This figure shows the influence of the quality of the subgrade on the service behavior of a calcium chloride stabilized road. Where the subgrade had a shear value of approximately 20 lb. per sq. in. at its yield point of 0.09 in., the 4½ in. of compacted stabilized gravel mat was in good condition. But on an adjacent area that had failed badly the subgrade showed a shear value of only 5.0 lb. per sq. in. at a deformation of 0.09 in. The quality of the stabilized mat was good as it had a shear strength of approximately 45 lb. per sq. in. The tests on the subgrade materials were made at the same depth (top 3 in. under stabilized mat) and within 12 ft. of each other.

The laboratory tests on these soil subgrades revealed some interesting results. Subgrade A, had a lower plastic limit of 19.5 and plasticity index of 16.3. The moisture content at the time the field shear tests were made was 21.4 per cent. Subgrade A-1, had a lower plastic limit of 20.7 and a plasticity index of 14.8. The moisture content at time of test was 18.3 per cent. Thus, for an increase of only 3.1 per cent in moisture content,

over this critical range, the stability was decreased 75 per cent. Subgrade A, had a moisture content of 10 per cent above its lower plastic limit, and in subgrade A-1, the moisture content was about 12 per cent below its lower plastic limit. These results emphasize the importance of keeping the moisture content in cohesive type subgrades below their lower plastic limit and the effect of rather small changes in moisture content, in the vicinity of this value, on the soil stability.

Figure 2 also shows graphically the

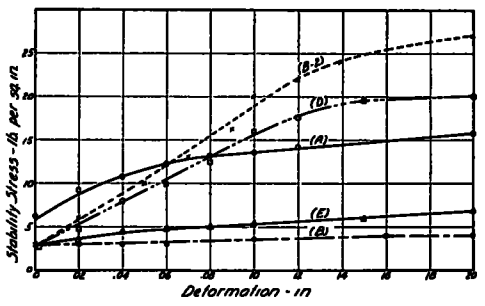


Figure 2. Project I-1. Subgrades Condition: (B) badly failed, (E) badly failed, (A) slightly deformed, (D) good

TABLE 1
SOIL TESTS ON SAMPLES FROM PROJECT I-1

Sample	Mechanical analysis: Passing ¾ in. Retained on ½ in.	Passing ½ in. Retained—No. 4	Passing No. 4 Retained—No. 10	Coarse sand No. 10—No. 40	Fine sand 0.42—0.05 mm.	Silt 0.05—0.005 mm.	Clay 0.005—0.001 mm.	Colloids Below 0.001 mm.	Physical characteristics: Moisture content	Weight per cubic foot (dry)	Liquid limit	Plasticity index	Shrinkage limit	Field moisture equivalent	Volume change	Lineal shrinkage	Specific gravity	pH value	P. R. A. Classification
									%	lb.									
A	7	5	2	1	41	21	12	11	16.3	106	29	11	16	26	19	6.0	2.60	6.2	A–2–4
B	0	8	6	7	15	37	13	13	24.6	91.8	50	26	12	33	41	10.8	2.66	7.4	A–7–6
B–2	2	3	4	4	18	33	18	18	13.0	106	41	20	12	29	34	9.5	2.66	7.4	A–7–6
D	3	29	14	16	15	14	6	3	13.0	.....	22	5	14	22	17	5,1	2.60	7.2	A–2
E	0	2	2	1	10	54	18	13	23.6	92.5	45	21	12	32	38	10.4	2.63	7.2	A–7

Average gradation of calcium chloride stabilized base, passing 1 inch 98 per cent, passing ¾ inch 79 per cent, passing ½ inch 63 per cent, passing No. 4 sieve 48 per cent, passing No. 10 sieve 33 per cent, passing No. 40 sieve 22 per cent and passing No. 270 sieve 10 per cent. Plasticity index 3.0.

relationship between shear strength of the subgrade and the service behavior of a well graded and low plasticity crushed stone calcium chloride stabilized base, 6 in. thick, covered with a 2½-in. open type bituminous wearing course. These tests were made in August 1939, about one year after the wearing course was placed. Table 1 contains the physical tests on these subgrades and on the stabilized crushed stone base.

The tests shown in Curve B-2 were made on the soil in the shoulder adjacent to the location of tests given in Curve B and at a similar depth. The increased shear strength shown in Curve B-2 is due to the lower moisture content in the subsoil of the shoulder. From the results given in Table 1, the moisture content in sample B-2 was only 62 per cent of its lower plastic limit while the moisture content of sample B was 103 per cent of its lower plastic limit. The shear strength of the B-2 subgrade was approximately seven times greater than that of subgrade B. These tests were made within 6 ft. of each other and the physical tests and classification show that the materials are similar. Table 1 also shows a difference in dry weight per

cubic foot of 14 lb. in favor of subgrade B-2.

The subgrade shown in Curve E had a moisture content equivalent to 98 per cent of its lower plastic limit and the

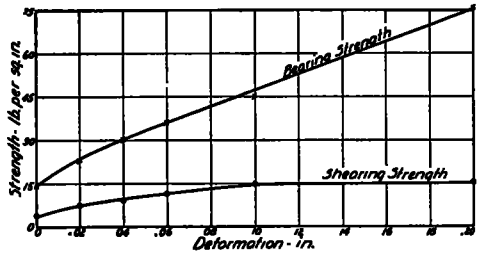


Figure 3. Relation of shearing to bearing strength Project MJ-1

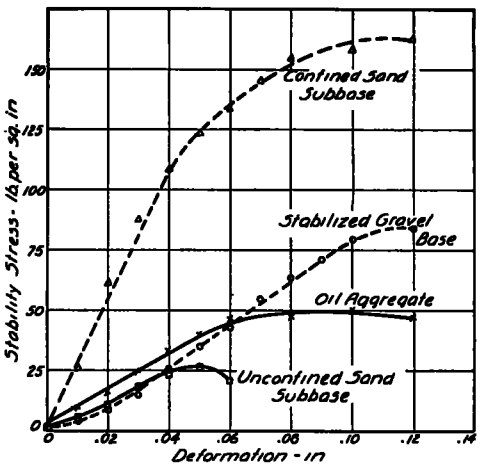


Figure 4. Project M-2

road structure was badly failed. Curve A represents a subgrade with a moisture content 91 per cent of its lower plastic limit and there was slight deformation noted in the road surface. The subgrade shown in Curve D had a moisture content equivalent to only 76 per cent of its lower plastic limit and the road surface was excellent. The shear strength of this subgrade would probably have been higher if it had been tested in a confined condition as the results given in Table 1 show it to possess a low plasticity index and a large quantity of fine internal friction type of material.

The failures in this road were due to a combination of unusual environmental conditions that permitted water to become entrapped in the high cohesive type subgrade, with the detrimental results shown quantitatively in Figure 2.

These two figures show that when the shear strength of a cohesive type subgrade is reduced to a certain value the service behavior of the road will suffer, regardless of how well the other components in the road structure are designed.

Relationship of Shearing to Bearing Strength of Cohesive Subgrades.

Figure 3 shows graphically the relationship of the shearing strength obtained by this machine to the bearing strength of cohesive type soils. The same size and shape plate was used in making these tests. The plate was parabolic in shape and had an area of approximately 14 sq. in.

The results of this test and others show that the unit bearing capacity of cohesive soils is approximately three times that of the unit shear strength obtained by this machine, when the tests are made with plates having the same shape and area. The relationship is based on the strength in shear at its yield point. The effect of different size and shape plates has not been studied.

Interrelationship of Stresses and Deformations in a Three Component Road Structure, When Each Component Is Tested Individually.

Figure 4 shows graphically the stress and strain results of the individual elements in a three component road structure.

The SC-4 oil aggregate wearing surface was $2\frac{3}{8}$ in. thick and was 5 months old at the time of test. The temperature of the

oil mat at the time of test was 62° F. at a depth ¾ in. below the surface.

The rounded gravel stabilized base was 5½ in. thick and had a weight per cubic foot of 150.4 lb. The moisture content at time of test was 3 per cent.

The sub-base was a sand lift 9 in. thick over a heavy clay subgrade. The sand lift material had a gradation quite similar to that of a fine concrete sand and the moisture content at time of test was 5 per cent.

The most interesting stress and strain relationship in this figure is that shown between the oil aggregate wearing surface and the stabilized gravel base. The yield point of the oil aggregate surface is approximately 0.06 in. and the shear strength at this point is 45 lb. per sq. in., which indicates a very good quality. The yield point of the stabilized gravel base is 0.1 in. and it possesses a shear strength of 80 lb. per sq. in. at this point. But this value of 80 lb. per sq. in. cannot be utilized for design purposes, as it is attained at a deformation of 0.1 in. which is greater than the yield point (0.06 in.) of the oil aggregate mat. That means for basic design purposes the shear strength of the stabilized gravel base at or slightly below the yield point of the oil aggregate surface, should be used. In the present case the strength of the base at this point (0.06 in.) is approximately 45 lb. per sq. in., which can be classed as good. A similar analysis of the relationship between the stabilized base and the confined sub-base can be made.

The characteristics revealed by these two curves indicate that it is possible to have a surface failure and not a base failure due to loading. A wheel load of such intensity as to cause an 0.08-in. movement in the base would not cause it to fail as this movement is less than its yield point, but this deformation is greater than the yield point of the oil aggregate surface, and small cracks would be expected to appear. In general, this figure shows that by knowing the characteristics of the individual components, their interrelationship can be determined and the service behavior of the entire road structure can be predicted.

Strength of Overlying Components Greatly Increase the Stability of Frictional Type Subgrades.

In Figure 4 is shown graphically the stress and strain results of the sand sub-base tested in an unconfined and a confined condition. The unconfined tests were made on the sand sub-base after removing the oil aggregate surface and stabilized gravel base. The confined tests were made by testing the sand sub-base with the overlying components in place and acting as a surcharge.

The results show that the unconfined sand sub-base had a shear strength of 25 lb. per sq. in. at a deformation of 0.04 in. For a similar deformation the confined tests revealed a shear strength of 110 lb. per sq. in. In other words the confining action of the overlying stabilized gravel base and the oil aggregate surface increased the shearing strength of the sand sub-base approximately five fold. The effect of this surcharge appears to be a function of the inherent strength of the overlying layers rather than of their weight.

A sand lift should be so constructed that it extends across the full width of the road grade and should never be placed in an undrained trench.

In general this figure shows that the use of sand lifts over heavy soils is probably the most effective means of obtaining high supportability and at the same time cutting off the detrimental capillary action of cohesive type subgrades.

Strength of Overlying Components Only Slightly Increase the Stability of Cohesive Type Subgrades.

Figure 5 shows graphically the stress and strain results of an unconfined and

a confined clay loam subgrade along with that of the overlying calcium chloride crushed stone stabilized wearing course. The results show that the shear strength of the confined cohesive subgrade is only about 10 lb. per sq. in. stronger than the unconfined subgrade. In other words, the confining action of the 50 lb. per sq. in. stabilized crushed stone wearing course

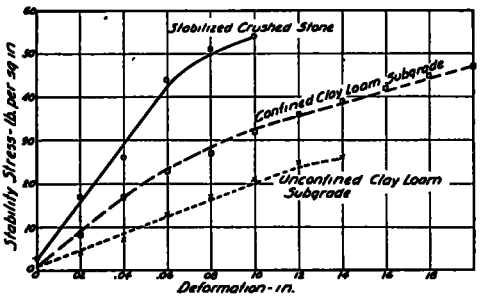


Figure 5. Project W-1

only slightly affects the shear strength of the cohesive type subgrade.

It is possible that as the cohesive type subgrades decrease in shear strength, such as under certain conditions in the spring, that the overlying layers will have no beneficial effect. Such a condition would be one in which the yield point of the subgrade would be greater than that of the overlying layers and the applied load would have a tendency to compact rather than displace the subgrade vertically.

A Set of Tests Revealing an Ideal Stress and Strain Relationship Between the Base and Wearing Course.

Figure 6 shows graphically an ideal relationship between the stress and strain characteristics of a calcium chloride stabilized gravel base and an oil aggregate surface. This figure shows that the stress and strain relationship of these two components are almost identical up to the abnormally high shear strength of 115 lb. per sq. in.

The oil aggregate surface contains an SC-8 oil and was 5 months old at time of test. The total thickness was 2½ in. and the temperature of the mat was 60° F. at time of test. The calcium chloride stabilized gravel was 5½ in. thick and contained 38 per cent of crushed material.

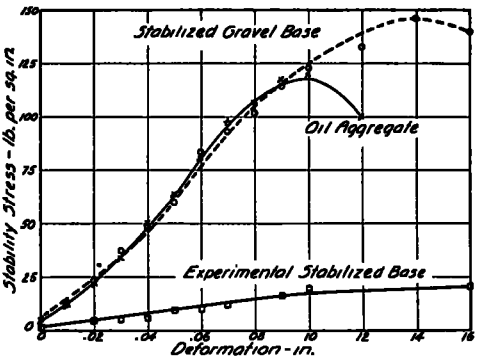


Figure 6. Project M-3

The moisture content at time of test was 4 per cent.

The shear strength of 120 lb. per sq. in. for the oil aggregate surface is the highest strength ever obtained for this type of mat, and the 145 lb. per sq. in. is the highest value obtained to date on a calcium chloride stabilized road surface or base.

In general this figure shows what can be attained in the way of both high quality and similarity in structural properties between the components in a road structure of this type. The lower curve in this figure contains the results obtained on a short experimental section in which a light bituminous material was mixed in with the regular stabilized material. The oil aggregate surface placed over this base failed badly. The results given in Figure 6 show the lack of sufficient stability. This stability was probably less in the summer due to the higher temperature (absorbed heat), as it was at this period that the surface failures occurred.

CONCLUSIONS

The results obtained by this machine show:

1. That to fully evaluate the significance of the stability factor of each component, it is necessary to obtain the stress and deformation data along with the moisture content and density of the structure at the time of test. These tests should be supplemented by the gradation and regular soil constant tests, as there is some indication that a soil factor may be derived from a combination of these tests which is related to the stability factor.
2. That the relationship between the stability of confined and unconfined subgrades of the frictional and cohesive types is widely different, the confining action affecting the strength of the cohesive types tested only slightly, but greatly increasing the strength of the frictional type.
3. That the stability strength obtained by this machine on cohesive soils is related to the bearing capacity. The bearing capacity being approximately three times the stability strength.
4. That based on the service behavior of stabilized mats of normal thickness, the cohesive type subgrades may be classed as follows:

Shear strength (lb. per sq in.)	Service behavior
Over 30	Excellent
Over 20	Good
Between 20 and 15	Good to fair
Between 15 and 10	Fair to poor
Below 10	Very poor

For the frictional type subgrades the confined tests gave the better indication of their true worth and the surcharge effect of the overlying layers appears to be a function of their inherent strength instead of their weight.

5. That a thorough study of the stress and deformation relationships of the different components that make up a non-rigid type road structure offers a very promising field for the development of a design method, as from the characteristics of the individual components and their interrelationships, we may be able to predict the performance of the road structure as a whole. The results indicate that only the stress developed in the stabilized base and subgrade at or slightly below the yield point of the wearing surface should be used for designing purposes.

The machine provides a means of eliminating incorrect deductions for failures.

It also is a means of obtaining quantitative data which when interpreted will lead to the elimination of the present unsatisfactory qualitative term—stabilization.

Field testing offers the quickest and most practical means of getting information from which some type of a design method may be formulated, as here you have the actual evidence of the behavior of the road structure under the forces of traffic and weather.

ADDITIONAL INFORMATION NEEDED

Before a tentative design method can be formulated for stabilized road structures, it will be necessary to:

A. Acquire more data similar to that included in this report, which correlates the structural qualities of each component to the service behavior of the road during varying climatic conditions.

B. Correlate the shear test results of the base and wearing course to their ability to distribute the imposed wheel loads over larger areas of the subgrade. This also includes the factor of thickness of these layers. The present work on load distribution being carried on by Iowa State College and the National Crushed Stone Association will undoubtedly produce valuable information.

C. Determine quantitatively the values of layers of different thicknesses of frictional material, such as sand lifts, on heavy soils for increasing supportability of stabilized mats and for breaking the detrimental capillary action of the cohesive soils. The Calcium Chloride Association through a Research Fellowship at Michigan State College expects to study this problem during the present year by a series of circular track tests.

Information is also needed on the correlation of the results obtained by this machine with those determined by the laboratory tri-axial loading method. In such a case the quantitative effect of such variables as, moisture content, density, "seasoning," and other structural variables could be controlled and determined. It is possible that the field testing apparatus may be utilized as a laboratory machine for testing stabilized soil slabs, in which the above mentioned variables could also be controlled.

Based on experience obtained during the inspection of several hundred miles of flexible type roads in "spring breakup" periods, there appears to be a need for more information on why the light (less than $\frac{3}{4}$ in.) bituminous surface treatments are more susceptible to surface failure than are the heavier (over $1\frac{1}{2}$ in.) types on similar stabilized bases.

On the roads which are covered with the lighter bituminous mats, the upper inch or so of the stabilized base directly under the bituminous mat softens and this causes the thin surface mat to deform, crack and sometimes peel off. This is not the case with the heavier bituminous mats.

The following factors in the problem apparently need to be studied:

A. Are the thin bituminous mats pervious and do they permit water to enter either during the fall rains or during the spring thaws? Or is this an accumulation of surface and capillary moisture? Are the thicker mats impervious to moisture?

B. Are the thin bituminous mats insufficient insulating layers and do they so regulate the rate of freezing as to cause a concentration of fine ice crystals in the upper layer of the stabilized base? What are the insulating properties of the thicker bituminous mats?

C. Do the thin bituminous mats absorb the heat of the sun to such an extent as to cause rapid thawing of the upper layer of the stabilized base and does this plus the vibratory action of traffic cause the semi-plastic condition? What is this effect on the thicker mats?

D. Does the structural strength plus the increased thickness of the thicker mats distribute the loads over a larger area of the base and thus produce more favorable results?

STATUS OF SOIL STABILIZATION IN THE UNITED STATES—DECEMBER, 1939

By W. H. Mills, Jr.

Testing Engineer, South Carolina State Highway Department

SYNOPSIS

This report is a summary of answers to a questionnaire sent to all State highway departments and covers work done since 1924. Of the 47 States answering, all but 6 have performed some stabilization work, which has been defined as the alteration of soil by the addition of other materials. The types of stabilization are divided into 10 groups as follows: soil-aggregate, RC type cutback, MC type cutback, SC type cutback, calcium chloride, sodium chloride, lignin binder, emulsified asphalt, tar, and portland cement. One table and 4 graphs are used to present statistical data showing the miles of each type constructed by States and by years and the totals to date. It is noted that the mileage of the soil-aggregate type greatly exceeds the total of all other types. In general, the various types are distributed throughout the country depending upon the soil conditions and the cost of the admixture. No data were received from municipalities, counties, or private concerns.

In compiling a summary of stabilized roads, the question of what roads should be included is paramount because of the very vague understanding of the term "stabilized roads" and also because of the application of this term to such varied types of construction work. It was decided to confine this summary to those roads in which the soil had been changed by the addition of an admixture. Accordingly, the questionnaire sent out requested a report of the mileage of all types of stabilized roads which would conform to the following definition: "A stabilized road may consist of subgrade, base, or surfacing mixtures in which unsatisfactory local soil, either as found on the project or hauled in from nearby pits, is altered by the addition of some other material such as soil, aggregate, bituminous materials, chemicals or portland cement so that it will support the loads applied to it by traffic."

This definition is intended to include only roads in which local soils have been treated by the addition of other materials. As compared with natural mixtures of sand-clay, clay-gravel and similar materials the distinction is purely arbitrary as these materials when used correctly certainly provide a stabilized road. The definition is intended to eliminate naturally mixed materials such as the limerock in Georgia and Florida, caliche in Texas, sand-clay-gravels in the New England States and the Middle West and the sand-clays or top-soils found in the Southeastern States unless these materials are combined with some admixture.

Information has been obtained through questionnaires answered by 47 of the highway departments, the National Park Roads Division of the Public Roads Administration and the U. S. Forest Service. An attempt was also made to secure information from the offices of the Chief of Engineers, U. S. Army, the Works Projects Administration, and the Public Works Administration; but these agencies reported that they did not have the information readily available.

No attempt has been made to include stabilized roads or airports constructed by private individuals, municipalities or other political subdivisions because of the impossibility of obtaining the necessary information in the short time available. It is well known that a large amount of stabilization work has been performed by such agencies and the projects should be included in order to obtain a complete picture of the stabilization work which has been performed. Several

producers have furnished valuable information which shows that the projects constructed by the state highway departments constitute only a part of the total stabilization work performed to date. With some materials, less than half the totals reported by the producers has been under state supervision.

In the questionnaire the types of stabilization were separated into subdivisions depending on the type of admixture and information was requested on the total mileage of each of these types constructed during the period 1925-1929 and each year thereafter. This grouping was selected in order to obtain data on the trends in this type of work, especially during the last few years. Information on the types of soils stabilized and on the intended use of the stabilized mixture was also requested.

According to the reports received, very little stabilization work was performed prior to 1929, but since that time there has been a steady increase. Five states have not yet constructed any stabilized roads on account of an abundant supply of suitable local material, but all the other states reporting have constructed at least some of the sections included in this definition. Many of them consider that the work performed to date is experimental and does not warrant definite conclusions at this time. This is especially true where admixtures other than local aggregates have been used. In other states, however, the feeling appears to be that sufficient background has been obtained to justify the construction of certain types as a part of the regular program.

Most of the stabilization work has been the treating of soils in place to form a base to which a relatively thin bituminous wearing course is applied but there are several instances where the subgrades for high type pavements have been treated and there are also some types of stabilized roads such as the soil aggregate mixes and most of those treated with calcium or sodium chloride to which no wearing surface is added. In certain parts of the country it is common practice to wet the subgrade to approximately the field moisture equivalent immediately prior to the laying of concrete pavement but this type of soil treatment is not included in this summary.

Soils of every classification have been treated, from pure sands to the most plastic clays. Stabilization of sand soils consists of adding a binder material but with clay soils the admixture must change the clay either by waterproofing it, binding it together or improving the grading. The type of soil and the cost of the various admixtures have the greatest influence on the type of stabilization selected.

The total mileages of individual types of stabilization are given in Table 1. Curves summarizing this information for each year are shown in Figures 1 through 4. Points on the curves were obtained by adding the mileage each year to the previous total.

Curve 1 of Figure 1 shows the total mileage of soil and aggregate type stabilization constructed during the different years to date and curve 2 on this same figure shows the total of all other types of stabilized roads. The reason for showing these curves separately is because the soil-aggregate type is often not considered stabilization as it is distinctly different from stabilization by the addition of a bituminous material, portland cement or chemical. The tremendous mileage of naturally mixed soil-aggregate combinations used as blanket courses or bases for all types of surfaces is, of course, not shown as they are excluded by this definition. However, the mileage of soil-aggregate roads exceeds by far the total mileage of the other types. This is quite natural and consistent with one of the first principles of good engineering

TABLE 1

SUMMARY OF STABILIZATION PROJECTS CONSTRUCTED 1925 THRU 1939

State	Soil or aggregate	Type of stabilization (Length in miles)											
		Asphaltic materials				Chemical							
		MC	RC	SC	Other	Calcium chloride	Sodium chloride	Lignin	Other	Portland cement	Tar	Emulsion
Alabama	....	...	23.0	...	...	10.0	...	...	...	...	...	20.0
Arizona	Naturally occurring stable materials used.											
Arkansas	....	...	....	0.5	...	....	...	...	...	4.6	...	17.4
California	Cut-Back Asphalt used each year, mileage not reported									8 0	...	23.3
Colorado	1154.0	...	....	...	...	....	...	...	...	...	...	...
Connecticut	No Stabilization (Natural mix Sand & Gravel plentiful)											
Delaware	No Stabilization (Natural mix Soils used)											
Florida	807.0	...	443.0	...	...	....	...	...	..	..	69.5	...
Georgia	441 0	9 0	....	....	...	12.0	...	...	..	15.0	51.0	25.0
Idaho	....	.	....	...	...	....	...	130.0	..	...	...	1.0
Illinois	No report received.											
Indiana	372.4	...	....	...	...	140.5	52.6	...	2.2[c]	...	...	...
Iowa	220 3	3.9	....	4 3	...	1.8	...	...	..	4.2	2.5	10.5
Kansas	76.7	...	....	...	...	8.0	8.8	...	..	2.0	...	3.4
Kentucky	....	1.0	....	35.1	...	11.2	...	...	..	6.5	1.0	20.0
Louisiana	1598.7	...	....	...	...	....	...	...	..	...	...	...
Maine	....	...	....	...	...	38.0	...	...	..	...	...	1.0
Maryland	62.0	29.8	....	...	...	27.5	3.8	12.0	..	3.8	15.7	...
Massachusetts	....	.	1.0	...	...	....	...	...	..	0.9	...	0.5
Michigan	....	...	....	...	. .	91.0	100.0	...	31.0[e]	1.3	...	...
Minnesota	277.0	...	.. .	...	. .	498.0	...	..	..	...	...	...
Mississippi	487.5	...	34 0	...	...	5.0	2.0	...	..	24.0	...	8.0
Missouri	256.0	28.4	....	122.6	. .	17.0	...	...	..	12.9	16.4	20.4
Montana	7 0	2.0	....	3.3	...	....	...	...	..	...	...	8.0
Nebraska	784.0	11.0	...	..	327.0[b]	36.0	....	...	.	10 0	4.0	4.0
Nevada	Gravel used in Stabilization each year, mileage unreported											2.5
New Hampshire	....	...	....	...	...	94.8	...	...	..	...	...	...
New Jersey	....	..	....	...	...	....	...	...	..	3.2	...	...
New Mexico	No Stabilization (Various Asphalts used for surface only)											
New York	....	...	....	...	...	213.0	4.4	...	..	10.3	272.7	...
North Carolina	482.4	...	79 0	...	...	....	...	...	..	7.2	46.9	1.2
North Dakota	....	1.0	(Gravel used in Stabilization each year)							...	...	1.0
Ohio	....	.	....	...	...	71.3	2.4	...	..	5.0	...	6.6
Oklahoma	84 0	1.3	....	18.8	...	2.4	2.4	...	..	6.1	...	31.4
Oregon	...	...	....	...	...	....	...	...	..	...	...	2.5
Pennsylvania	38 6	...	....	...	. .	70.2	23.6	...	..	1.9	...	...
Rhode Island	No Stabilization, Local gravel plentiful											
South Carolina	21 6	1.4	718.5	27.2	1.4[f]	....	...	...	..	71.3	64.2	1.8
South Dakota	No Stabilization											
Tennessee	54.8	0.7	0 7	...	...	4.1	..	...	.	1.5	1.4	1.5
Texas	[g]	58.4	196.5	457.6	...	0.6	7.8	...	..	27.5	...	56.3
Utah		...	....	..	...	....	...	0.9	..	...	...	...
Vermont	....	...	....	...	...	1 5	0.3	...	.	...	0.5	0.5
Virginia	....	4.0	0.5	...	...	40.5	14.0	...	.	4.8	...	3.8
Washington	....	..	....	..	...	....	...	0.8	..	1.0	...	...
West Virginia	....	5.8	8.0	...	...	8.8	...	...		...	14.0	8.1
Wisconsin	13.6	...	....	...	...	....	...	...	..	3.3	...	...
Wyoming	....	..	....	50.0	...	....	...	...	..	...	...	...
Nat. Park Rds.[a]	....	10.9	....	..	...	43.3	...	...	.	11.6	12.7	35.9
Forestry Service	1312.0	1 2	11 4	29 5	...	34.8	11.3	25.8[d]	..	0.8	...	...
Totals	8550.6	169.8	1515.6	748.9	328.4	1481.3	233.4	169.5	33.2	248.7	572.5	315.9

Remarks: [a]Of Public Rds. Adm. [b]Unclassified Cut-Back Asphalt. [c]Natural brine. [d]By-product of paper mills—similar to lignin. [e]$MgCl_2$. [f]Colprovia process. [g]Soil-aggregate stabilization each year, approximately 500 mi. built to date.

for a good engineer uses the materials available in the most economic and efficient way. The curves show a steady increase in the use of both methods of

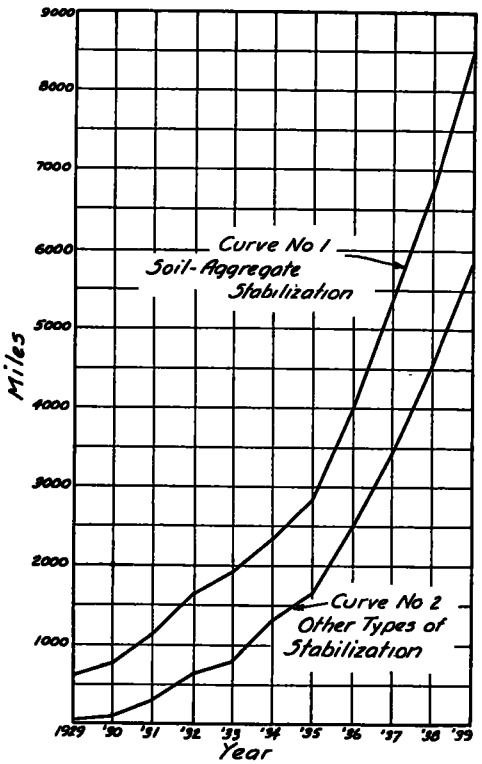


Figure 1. Soil-aggregate stabilization compared with all other types. Cumulative graphs

used. One of the striking facts in comparing cut-back asphalt is that, in various sections of the country, the engineers appear to prefer a certain type of cut-back to the comparative exclusion of others. For instance, rapid curing cut-back asphalts are used almost exclusively

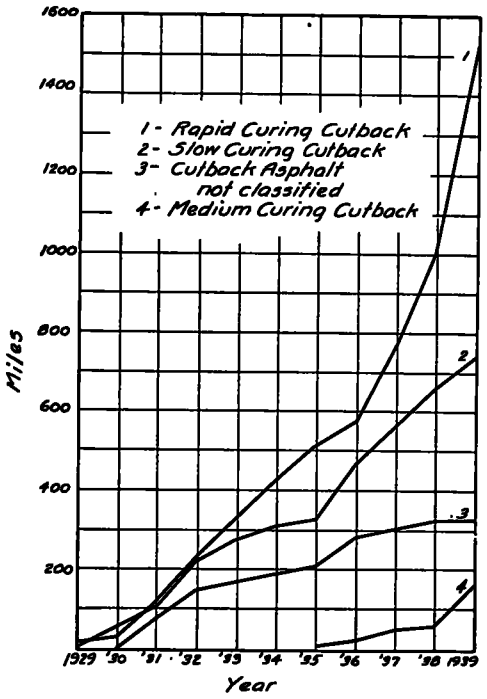


Figure 2. Miles stabilized with different cutback asphalts. Cumulative graphs

stabilization with much greater increases during the last four years. The aggregates which are used consist principally of sand and gravel although crushed stone, stone screenings, mine tailings and similar waste materials have been used successfully. The controlling factor is the availability of the material. The grading of the mixture is adjusted by the addition of these aggregates so that the mix will be stable with a clay content sufficient to bind the particles together but which will not soften in wet weather.

Figure 2 shows the mileages in which cut-back asphalt materials have been in the southeastern States whereas the slow curing type is used West of the Mississippi River. The great increase shown for RC type cut-back asphalts in the last three years is the result of work in the southeastern States in combining this material with sandy soils. California reports that "any soils which can be satisfactorily mixed with cut-back asphalt are not considered stabilization." This state has used great quantities of SC and MC oils but none of this mileage was reported.

Figure 3 shows the mileages of roads to which chemicals of different types

have been added. This type of stabilization is largely confined to areas having an abundant supply of coarse aggregates, particularly gravel, and the greatest mileages of this type are to be found in the New England States and States adjacent to or near the Great Lakes. All of this work has been performed since 1934 and the use of calcium chloride has increased very rapidly.

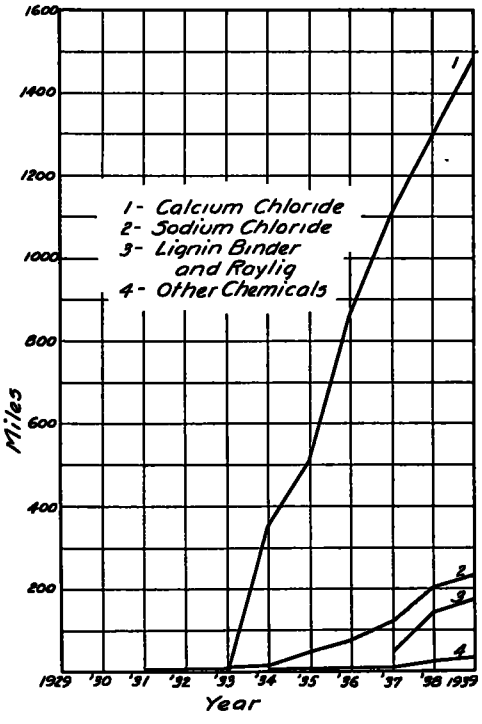


Figure 3. Miles stabilized with chemicals. Cumulative graphs

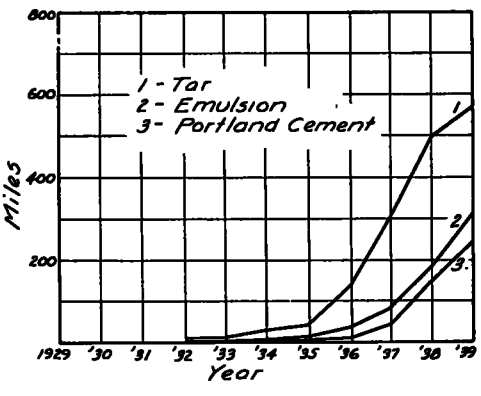


Figure 4. Miles stabilized with Tar, Emulsion and Portland Cement. Cumulative graphs

Figure 4 shows the mileages stabilized with emulsified asphalt, tar and portland cement. Of course, there is no special reason for grouping these materials together except that they do not fit into either of the classifications just discussed. Work with these materials has developed since 1932 and the increases since 1936 have been fairly rapid.

Table 1 gives the totals of each type of stabilization constructed by each state from 1925 through 1939. This tabulation shows the distribution of the different types.

1. The soil-aggregate type is reported by 19 States, many of which have constructed several hundred miles.

2. Medium curing cut-back asphalt has been used in 14 States—generally in short sections as only two report more than 25 miles.

3. Work with rapid curing cut-back asphalt is confined almost exclusively to the southeastern States and 76 per cent of the mileage reported is in two of the ten States in which it has been used.

4. Slow curing oils or cut-backs have been used in 9 States—mostly west of the Mississippi River. Only two States report more than 100 miles of this type but it is our understanding that great quantities of slow curing oils are used in California and other western States in work closely allied to stabilization.

5. Stabilization with calcium chloride is widely distributed, 23 States reporting its use. Three States report more than 100 miles, with one practically up to 500 miles.

6. Sodium chloride has been used by 12 States and only two report more than 50 miles.

7. The use of lignin binder in soil sta-

bilization is practically confined to one State as this State reports 77 per cent of the total.

8. Portland cement in soil work is well distributed. Mileage in which this material was used is reported by 24 States. The sections are, however, generally short and only two States report more than 25 miles.

9. Tar has been used in 13 States; approximately 50 per cent of the mileage is in one state but several report more than 40 miles.

10. Stabilization with asphalt emulsion is also well distributed, being reported by 26 States, but the sections are generally short and only two States report more than 25 miles.

11. Data submitted from the U. S. Forest Service shows that they have used practically all of the methods of stabilization mentioned above. Their work is scattered in different parts of the country and most of the sections are short. The National Park Roads Division of the Public Roads Administration has constructed sections using several of the methods contained in this report. Their work is widely distributed and is confined usually to short sections.

SUMMARY

It must be emphasized that the information may be somewhat indefinite due to variations in the interpretation of the definition of a "stabilized road." For instance, several of the States report hundreds of miles of bases stabilized with naturally mixed gravel, sand-clays and similar materials which are not included in this summary as the work is not considered to be within this definition. This is particularly true of base and blanket courses but any naturally mixed materials which have been used are not included. The sum of these mileages would be several thousand. Mention of such construction was made by Tennessee, North Dakota, South Dakota, Nevada, Utah, Connecticut, Delaware and Rhode Island and no doubt most of the States use construction of this type quite extensively.

In addition, there has been some confusion concerning the inclusion of the oil type road developed in the west, particularly in California. It is understood that thousands of miles of this type have been constructed in California and that there is considerable mileage in Colorado, Idaho, Montana, Nevada, Oregon, Utah, Wyoming and New Mexico. This type of construction is not included in the summary because it would be almost impossible to obtain accurately the entire mileage involved and also because there is some question as to whether this type of road fits in with the definition of stabilization which is used herein.

The information presented does show, however, a definite trend in stabilization work. There have been great increases in the rate of construction of these particular types in the last few years. The total mileage is believed, however, to be only a fraction of the total mileage of highway construction performed during this period. During 1937, for instance, information from the American Association of State Highway Officials shows that 30,632 miles were constructed by the 48 highway departments. This summary shows that 2410.9 miles or 7.9 per cent was stabilization work. Similarly in 1938 the mileage of stabilized roads was 8.2 per cent of the total constructed. These figures, while not particularly impressive, do indicate that soil stabilization is taking a place as a distinct method of road building to supplement other types.

DISCUSSION OF STATUS OF SOIL STABILIZATION

MR. C. N. CONNER, *Public Roads Administration:* I would like to ask Mr. Mills if his survey of binders used in soil stabilization showed that best results with calcium chloride or other salts are limited to areas of relatively high humidity.

MR. W. H. MILLS, *South Carolina State Highway Department:* The information shows that the States near or adjacent to the Great Lakes or the New England States were the principal users of calcium chloride.

MR. C. L. MCKESSON, *American Bitumals Company:* Mr. Mills' report deals principally with work of this type constructed by State highway departments, and I believe that the picture will be made more complete by the inclusion of some data regarding stabilization work by other organizations.

My knowledge pertains principally to soil stabilization in which emulsified asphalt has been used as a stabilizing medium. The first project of this type was an experimental section constructed at Clover Field, Santa Monica, California, in 1929.

The first sizable project of soil stabilization with emulsified asphalt was at the Glenn Martin Airport, in Baltimore, Maryland, in 1930. In 1939, the Glenn Martin Company undertook a large program of airport runway construction and again selected emulsified asphalt stabilization. This is a significant indication of the satisfactory performance of the first large stabilization project.

My record of projects includes a large number of State highway jobs, but I have no way of determining how many of these have been included in Mr. Mills' data, and I have, therefore, omitted my figures on State highway projects. I have, however, a lengthy, although incomplete, list of jobs not on State highway systems which will doubtless be of interest to those following this type of construction.

Projects constructed by Governmental Departments, including the Army, Navy, Public Roads Administration and National Park Service, aggregate 1,825,266 sq. yd.

Jobs constructed by municipalities and counties aggregate 5,664,184 sq. yd.

Projects constructed privately and without public funds aggregate 769,276 sq. yd.

My tabulation includes only projects with which I have had personal contact and is, therefore, far from complete. The projects here reported, if converted to a 20-ft. road surface, would total over 720 miles, exclusive of all State highway projects.

DR. C. D. LOOKER, *International Salt Company:* Mr. Mills' report indicates limited use of salt by State highway departments, which does not make a true picture of situation. The bulk of salt stabilization this year has been in counties, townships and municipalities with at least 250 miles in New York, 250 miles in Michigan, 30 miles in Kansas and the Municipal Airport in Hutchinson, Kansas and some additional mileage in Rhode Island and elsewhere.

SOIL STABILIZATION WITH EMULSIFIED ASPHALT IN THE MID-CONTINENT AREA

By B. I. Scoggin

Col-Tex Refining Company

SYNOPSIS

Stabilization with emulsified asphalt in Texas, Oklahoma, Iowa, and Nebraska is described. Costs on this work have ranged from $0.22 to $0.28 per sq. yd. for a 4 in. compacted base. These costs obtained on 200 mi. of an equivalent 18 ft. roadway. The author expresses the opinion that while high density in the finished construction is commendable, the obtaining of this is often an expensive luxury. The chief reason for the use of bituminous material in stabilization is to make the resultant mixture more resistant to the effects of moisture. No failures have been noted in the projects that were compacted while below the optimum moisture content. It is deemed advisable to dry the stabilized base to below 5 per cent, and 3 per cent is advisable, as the presence of water aids the entrance of more moisture into the mix. It is recommended that the stabilized mix be compacted in relatively thin layers.

This review lists emulsified asphalt soil stabilization projects in Texas, Oklahoma, Iowa and Nebraska, with a total area equivalent to 225 to 250 miles of 18-ft. roadway. (See Tables 1, 2 and 3.)

The term "Soil Stabilization" has been used to define practically all types of construction, wherein a soil or an aggregate not ordinarily satisfactory for construction purposes, is either mechanically or both chemically and mechanically changed in character from a waste material to a desirable building material. All soil stabilization has as its main objective, the reduction in cost of construction over the usual or conventional methods of construction. Table 1 lists base construction projects in Texas and Oklahoma. It is notable that treatment of a 4-in. depth of base has been accomplished for as low as 22 cents per sq. yd. (This is total cost, in place.)

This discussion is limited to the projects that are of special significance, from either a cost or a technical viewpoint and as related to the following conclusions:

1. Maximum dehydration is a more important factor effecting durability of soil stabilized by the use of emulsified asphalt than maximum density at time of compaction.
2. The quantity of emulsified asphalt necessary should be determined by test mixtures of the emulsion to be used and representative samples of the soil to be treated, following the efficiency requirements designated by McKesson (10,000 lb. minimum stability in lower ½-in. after 7 day absorption), or other requirements known to provide adequate control.
3. Four-inch compacted depth should be the minimum thickness. If the base is placed on a subsoil of high plastic index such as A-6 or A-7 type, the stabilized base should have a minimum thickness of 6 in.
4. Control tests should be made on the finished work to guarantee that all material placed will meet minimum requirements of efficiency test. (From a practical standpoint, this may require 1 per cent more emulsion than is indicated by laboratory test, since it is rarely possible to have field control as accurate as laboratory control).

The term "Soil Stabilization" is used to refer to treatment of any type material

TABLE 1

COSTS ON TYPICAL EMULSION STABILIZED BASE JOBS

Actual or bid cost[1] per sq. yd.	Equivalent cost per mile 18 ft. wide	Thickness compacted	Emulsion quantity	Aggregate	Type of mixing	Date constructed	Description
$	$	in.					
0.28	2956.80	4	4%	Soil from pit average haul 2.5 mi.	Stationary plant	1938	Texas SHD, Lee Cty., U. S. 77, Giddings to Lincoln, 4.5 miles.
0.276 (0.235)[2]	2914.56 (2481.60)	4	Min.[2]Contractorrequired to meet McKesson standard absorption and stability.	Gravel and soil from pits.	Stationary or travel plant, alt.	July, 1939	Texas SHD, Lee Cty., U. S. 77, Giddings-Fayette County Line, 7 miles.
0.248	2618 88	4	4.5%	Soil in place blended with blow sand from pits.	Stationary plant.	1938 and 1939	City wide, street paving. Midland, Texas.
0.262	2766.72	4	4%	"Blue shale" in place.	Blade.	1937	5 miles, between Rhome and Decatur, Texas. Highway U. S. No. 81.
0.640	6758.40	5	4% plus .5 gal. S. Y. bottom seal[3] and 1 gal. S. Y. topseal.	Mudshell-85% and soil-15% blend, hauled in.	Traveling plant.	1939	P. W. A. county job, 38 miles of various Nueces County Roads, Texas.
0 229	2418 24	4	3.5%	Soil in place.	Traveling plant.	1938-39	P. W. A. county job. 46.6 miles various county roads, Major Cty., Oklahoma.
0.222	2344.32	4	4.5%	Soil from various locations airport property.	Traveling plant.	1939	164,336 S. Y. Airport runways, Midland, Texas, Municipal Airport.

[1] Includes all costs of base complete, in place.

[2] Bid of .235 per S. Y. was received on stabilized base itself, but this contractor was high on preliminary work and topping, which was included in the same contract.

[3] Bottom seal consists of .25-.5 gal. per S. Y. sprayed on subgrade or mixed into bottom 1" of stabilized base. Usually costs .03 to .05 per S. Y. Used where there is high water table or bad subgrade.

which has more than 50 per cent passing a No. 40 sieve.

There is no denying the fact that compaction at optimum moisture content will give higher density, however, subsequent statements may seem to conflict with the established theories concerning compaction of aggregate and soil mixtures. They are offered with the hope that they will emphasize the relationship between actual application and the various moisture density theories.

TABLE 2

MOISTURE AND DENSITY TESTS MADE IN 1937, 1938 AND 1939, CASS-POTTAWATTAMIE COUNTIES

(Courtesy of the Iowa State Highway Department)

		Asphalt emulsion 9% water		Asphalt emulsion 9% water		Asphalt emulsion 6.6% water	
		Wt. lb.	%	Wt. lb.	%	Wt. lb.	%
October and November	1937	118.7	12.3	101.8	19.3	92.5	23.2
March	1938	115 4	11.8	106.0	15.5	103.2	18.3
June	1938	117 3	10.1	110.5	14.7	108.5	17.3
November	1938	114 5	11.2	105.6	16.2	101.9	18.6
May	1939	117.6	11.2	116.1	14.4	107.2	17.1
September	1939	.....	9.9	.....	14.6	.....	18.2

TABLE 3

LIST OF MAJOR CONSTRUCTION PROJECTS IN IOWA, TEXAS, OKLAHOMA AND NEBRASKA, WHERE EMULSIFIED ASPHALT STABILIZATION HAS BEEN USED

Sponsor	Location	Depth of base	Width	Size	Year built
		in.	ft.		
Iowa					
Iowa Highway Commission	P-39 Harrison Cty.	5	24	9.5 miles	1937
Iowa Highway Commission	P-144 Greene Cty.	5	24	5.2 miles	1937–8
Iowa Highway Commission	P-83 Cass-Pottawattamie	5	24	.9 Miles	1937
Town of Little Rock	Streets	4	..	7000 sq. yd.	1938
Town of Sutherland	Streets	4	..	56,000 sq. yd.	1938
Town of Soldier	Streets	4	..	18,000 sq. yd.	1938
Texas					
S. H. D.	Kaufman Cty.	4	24	6 miles	1939
S. H. D.	Kennedy Cty.	4	24	1½ miles	1937
S. H. D.	Lynn Cty.	4	34	3 miles	1939
S. H. D.	Eastland Cty.	5	3	12 miles	1939
S. H. D.	Hunt & Collins Cty.	4	24	5 miles	1939
S. H. D.	Hemphill Cty.	6	6 (shoulder)	7 miles	1939
S. H. D.	Gorman, Highway 67	4	22	1 mile	1937
U. S. Army Q. M. Airports.	San Antonio	4	..	300,000sq.yd.	1936–7
Oklahoma					
S. H. D.	Binger, Highway 41	4	20	5 miles	1937
S. H. D.	Norman-Moore U. S. 77	6	2 (shoulder)	9 miles	1937
S. H. D.	Oklahoma City, U. S. 270	6	2 (shoulder)	25 miles	1937
S. H. D.	State Highway 17, Grady Cty.	5	20	3000 ft.	1936
Nebraska					
S. H. D.	Crete	5	..	4.3 miles	1939
W.P.A. and City	Grand Island Airport	3	Runways	..	1936

In current procedures for soil stabilization, the terms optimum moisture content and maximum density are often emphasized to a point where one studying the literature on any type of bituminous stabilization, may get the impression that the item of most importance is to obtain density in the finished construction and that if high density is not obtained, the construction is a failure. There is no doubt that high density is commendable, but we believe that it is often an expensive luxury. The only excuse for the use of bituminous material for soil stabilization is to make it resistant to the effects of moisture. We add bituminous material, not to get density or maximum compaction, but to render the soil particles as nearly water-proof as possible. Of the 250 miles of stabilization reported here, there are no failures that can be attributed to the base being compacted while below the optimum moisture content. On the other hand, there are unsatisfactory projects which were compacted under the so-called optimum conditions.

Table 2 gives density and moisture content tests of a series of soil stabilization projects in Pottawattamie County, Iowa. These sections were compacted in 6-in. layers by the use of a sheepsfoot roller, which was used until the feet of the roller "walked" out of the mix, so that there was no chance for further compaction by that method. This rolling was accomplished as near as possible to the optimum moisture content. The moisture content of these bases is essentially the same now as when constructed two years ago, even though the compacted base was allowed to dry for several months before a prime coat was added. A thin prime coat was used to prevent abrasion of the surface but still did not prevent drying. There is no indication from the above tests that these base projects will ever dry. For soil stabilization with emulsified asphalt to be effective, it must be dried. We often say that the stabilized base must dry to below 5 per cent. It would be preferable to have them contain less than 3 per cent moisture when completed, as the presence of water aids the entrance of more water.

Some will say that if the base has been compacted to maximum density there is no space for water to enter, therefore, such a base is stable regardless of moisture. It is of course true, that a material that is compacted at high moisture content will not have high density if water content in excess of voids is present. It does not follow that density is the answer to stability for if such a conclusion were justified, there would never be any reason for the use of bituminous material for stabilization. When we recognize water-proofing as the paramount purpose in bituminous stabilization, we can arrange our construction procedure so that it will result in maximum water-proofing and stability.

In the 46 miles of stabilization mentioned in Table 1, as job No. 6, located in Major County, Oklahoma, the moisture content at the time of rolling was 5 per cent or less. The procedure outlined as follows would seem to be expensive since it requires several operations, but the total cost of 22 cents per sq. yd. does not bear this out, since this cost compares very favorably with those of other jobs in this vicinity. I think the reason for this will be shown in the following discussion.

CONSTRUCTION PROCEDURE, MAJOR COUNTY SOIL STABILIZATION

The soil on this job was a sandy soil with 90 per cent passing the No. 40 sieve with 15 per cent passing a 200 mesh sieve. The soil was mixed with 3 to 4 per cent bituminous emulsion and sufficient water for complete dispersion by

the use of a traveling machine known as the Woods Roadmixer. The completed base was 18 ft. wide, 4 to 4½ in. in depth.

Prior to the application of the emulsified asphalt, the roadway was scarified and bladed into two even parallel windrows. These were placed as closely together as possible and in the center of the road. The machine was such that it would straddle the windrows and by mechanical means, pick the material from the road, mix it with water and emulsified asphalt and re-deposit it in approximately the same position. To prevent the excess moisture in the mixed windrows from affecting the subbase immediately beneath the windrow, it was immediately bladed towards the edge of the road, leaving a layer 1 in. to 1½ in. thick, spread from the center to the side of the road. The operation of moving this windrow naturally assisted the dehydration so that when the moved portion reached the side of the road, it had very little excess moisture with less detrimental effect than if it had remained in the center of the road. The 1-in. to 1½-in. layer which was left on the subgrade was then allowed to dry from 24 to 48 hours, or until the moisture content was less than 5 per cent, during which time it was thoroughly rolled by a pneumatic roller. Care was taken to roll the 1-in. layer as soon as it was dry enough to bear the weight of the roller so it was compacted but not tracked.

While the 1-in. layer was drying and being rolled, the remaining soil remained in a windrow on each side of the road. To the casual observer, the outsides of the windrows were apparently drying very fast. Many of the engineers assisting with this work, were alarmed by the fact that the windrow seemed to be drying and would soon be too dry for compaction, but when the windrow was moved again, it was found that it had not dried appreciably, below 2 or 3 in. from the surface, so that as a whole, it was not only wet enough for compaction, but in some cases, was even too wet to allow immediate compaction after spreading.

As the windrows were moved from the side to the center of the road, approximately one inch of material was allowed to spread and was rolled while what was left of the two windrows was consolidated in the center of the road as one windrow of approximately the size of each of the original windrows. As soon as this second layer had dried the third layer was spread and rolled immediately. This last layer was handled with greater ease as this thickness is from 1½ to 2 in. Sometimes in the final stages of levelling this layer, we get an accumulation of approximately ⅛ to ¼ in. of dusty material on the surface. In order to prevent the loss of these dusty materials and to attach them to the materials below, it was sometimes necessary to sprinkle with a dilute emulsion (1 part emulsion—10 parts water) then blade or sweep into place with thorough rolling as the final operation. The total time consumed by all of the above operations would naturally be affected by weather conditions but during the 46 miles constructed by this procedure, four days was usually sufficient for completion of any section of the base. Rarely was over seven days necessary. At the end of this time, the entire 4 in. of base was compacted and contained less than 3 per cent moisture. The weight per cubic foot of the compacted material was 110 to 120 lb.

The contractor on this job was naturally interested in finishing with a minimum cost, so that at first he preferred to spread the mixed material in one 4-in. lift, instead of in three separate spreads. Spreading in one 4-in. lift might be economical if it were possible to get uniform and complete dehydration throughout the depth. Usually the fact that the

top drys faster than the bottom results in formation of cracks and fragments in the dried top layer, from movement of the top layer on the wet and unstable bottom layer.

Two weeks after laying the first mile of this road in one 4-in. lift, the contractor was still trying to spread it and roll it to a satisfactory grade. The top 3 in. of the 4 in. was then bladed into one windrow and relaid in layers each of which was rolled. Subsequent use of layer compaction and spreading resulted in the saving to the contractor of at least 50 per cent of the time which would have been required in the original 4-in. trial. The layer construction is preferable because it permits a contractor to mix, spread, dry and compact a base in from 3 to 10 days, whereas from 3 weeks to 10 weeks might be insufficient time for drying if the entire depth is laid at once. This is proven by the moisture test on the sections shown in Table 2, where tests after two years still show excessive moisture.

This method of drying and compacting has advantages other than speed of construction. For instance, the first 1-in. to 1½-in. layer acts as a bottom seal. This allows the bottom of the base to dry thoroughly, thus getting the maximum stabilizing effect of the emulsified asphalt which will prevent moisture from the subbase affecting the upper portions of the treated material. Multi-layered spreading and compaction will also tend to give a more even base.

There are those who object to this layer construction because there will be a plane of cleavage between layers that would cause *lamination* or *stratification.* These two terms are used in much of the literature on soil stabilization and this effect is said to be definitely something to avoid. I believe that those who have written of lamination and stratification have done so principally because usually the only time that this effect is noticed, is after there has been a failure in some particular base. This failure is usually due to the fact that when mixtures are placed in 4 to 6-in. layers, the bottom 2 or 3 in. seldom dry to a point where maximum stability is developed. When loads are applied to the road, the top which has dried, will transmit the load directly to the undried, unstable bottom layer. If the load is sufficient to cause movement, the movement will result in lamination. Naturally, if this continues, the upper layer will ravel, causing holes in the base. These holes and raveling are then attributed to lamination, where as, the lamination is the effect and not the cause, the cause being lack of uniform dehydration.

SUMMARY

For successful stabilization by the use of emulsified asphalt, design the base with a 4-in. minimum thickness and with sufficient emulsion to give a finished base meeting some known efficiency requirement. To provide for maximum dehydration before compaction depend upon subsequent traffic to increase density rather than upon weather conditions favorable to complete drying of 4 to 6-in. depth.

USE OF TAR IN BASE STABILIZATION

By T. F. Loughborough, *Construction Engineer, Virginia Department of Highways*
A. M. Miller, *Assistant Materials Engineer, West Virginia State Roads Commission*
N. F. Schafer, *Engineer of Maintenance, Indiana State Highway Commission*
A. C. Tilley, *State Engineer, Nebraska Department of Roads and Irrigation*
Arranged by Paul Rapp, *Public Roads Administration*

SYNOPSIS

The activities of the Indiana maintenance forces in constructing bases with a 55-80 viscosity tar are described by Mr. Schafer. Costs of this work varied from $0.21 to $0.37 per sq. yd. The compacted bases were 4½ to 5 in. deep and 18 ft. wide. It is estimated that under ideal conditions, 75 to 80 per cent of the cost will be for materials, that is, tar and chips; 5 to 10 per cent labor; and 16 to 20 per cent for equipment. Tack coat should be provided immediately after compacting to provide a temporary wearing course. This should be followed in about 10 days with a seal coat for the wearing surface.

Mr. Tilley describes Nebraska methods for stabilization with tar. A mix of 40 per cent selected soil, 60 per cent of coarse sand combined with 4 per cent of tar (RT3) was found to be an economical and stable mixture. Traveling pugmills were used and compaction secured with numerous pieces of equipment. It was found that after the moisture content of the mix has been reduced to about 6 per cent, the base was impermeable. A surface treatment of one-third of a gallon of RT 10 tar and 35 lb. of sand gravel per sq. yd. was applied after about a week.

Mr. Loughborough emphasized the careful consideration that must be given to the type of soil to be stabilized. He described two recently completed Virginia projects on which both tar and asphalt were used to stabilize a sand clay soil with a PI of 4.5. Wearing courses were also applied. Total cost of the construction was about $5300 per mile. Cost of the base stabilization was about $2900 per mile for the tar and $3250 for the asphalt.

West Virginia practice on 32 miles of gravel-soil-bitumen stabilization was described by Mr. Miller. Material from berms and ditches was added to material already on a 15-year old gravel road and mixed in a traveling pugmill with a cutback asphalt, asphalt emulsion, and tar as the binding or waterproofing agent. The cost of the stabilization with tar, done by contract, was about $3900 per mile or $0.37 per sq. yd.

INTRODUCTION

Loughborough: Not always, but usually, soils used for surfacing are found in low-lying or rolling sections of the country where grading is not heavy and quite often drainage is poor. Under such conditions a wide roadbed section should be constructed, with preferably a minimum width of 50 ft., sloping gradually from the ditch line to the crown of the road, with the slope of the shoulder flat enough to allow a car to go off on it in safety, and with the ditch line well below the bottom of the subgrade. Bringing the material in from the sides puts the roadbed higher than the surrounding ground, provides proper drainage and eliminates borrow, and although some failures can be expected on low cost surfaces these can be lessened to quite an extent if the foundation on which they are placed is properly drained and compacted. The ideal method although difficult to arrange because of the public demand for immediate hard surfacing, is to construct the roadbed and let it be subjected to traffic for a year or more before placing the surfacing.

Even a roadbed constructed by methods less exacting than those now required generally and put in use for a year or more before surfacing is ofttimes in better condition than one which has been constructed by the most careful methods and the surfacing material im-

mediately applied. Many benefits accrue, not only from compaction by time and weather, but by enabling the engineer to find and correct the weak places before the final surface is placed.

Schafer: Because of limited maintenance funds, a low cost all weather road would be a solution of the problem. If it is possible to use all the materials

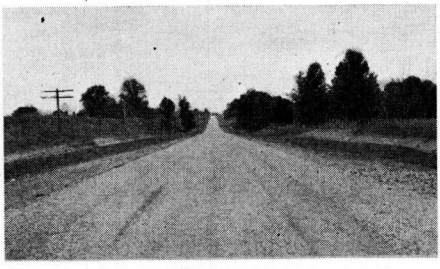


Figure 1. Loose Aggregate Surface Before Stabilization

available on the surface and add only an admixture, the result should be reasonable in cost.

Indiana has built stabilized bases for several years by using admixtures and new materials. This is satisfactory for new construction but is impractical and too expensive as a maintenance operation.

Tilley: Due to extensive mileage of low-revenue roads and the wide variance in the type of natural road-building soils, it has been necessary for the State of Nebraska to develop several types of low cost base courses and surfaces. The design and construction of the clay-gravel type of stabilized soil base courses and stabilized bituminous sand have been reported elsewhere.[1] The first stabilized soil base course in Nebraska using tar as the stabilizing agent was constructed in 1939 as an experimental project.

[1] "Stabilized Soil Base," Third Annual Montana Bituminous Conference, and "Stabilized Bituminous Sand," Eleventh National Asphalt Paving Conference.

LOCATION AND DESCRIPTION OF PROJECTS

Miller: During the 1939 construction season the West Virginia State Road Commission contracted 32 miles of gravel-soil-bitumen stabilized base course. Asphalt cutback, asphalt emulsion and tar cutback were used as the binding or waterproofing medium. The plan section provided for 5 in. compacted uniform thickness—18 ft. width.

The roadway to be stabilized was a traffic bound gravel surface constructed approximately 15 years ago. The average depth of gravel was $3\frac{1}{2}$ in. varying from 2 to $5\frac{1}{2}$ in. The original depth was 6 in. Suitable material secured from the berms and ditches was incorporated with the existing gravel surface to obtain the required depth.

The gravel-soil mixture during construction showed the following analysis:

Passing $1\frac{1}{2}$ in. sieve, %	99–100
Passing No. 4 sieve, %	30– 70
Passing No. 200 sieve, %	10– 35
Silt, %	6– 24
Clay, %	7– 17
Colloids, %	2– 9
Liquid Limit	19– 33
Plastic Index	0– 15

The subgrade varied from a clay loam to clay—B.P.R. Classification A-4 and A-7.

The bitumen and water content required for waterproofing and to obtain maximum density of the soil-gravel mixture was determined in the laboratory by means of the cell test apparatus, extrusion test, capillary and absorption tests and freezing and thawing. It was not deemed feasible to apply formulae for bitumen determination such as is used in the western States due to the high 200 mesh and colloidal content.

Schafer: In the spring of 1939, three roads in Indiana were selected for stabilizing those materials which were in place. Road No. 1 was 3.65 miles

long, part of which was traffic bound aggregate surface on new locations and the other was old county road which had been widened. The aggregate varied from 1 to 4 in. in depth and the subgrade was sandy loam and clay. Several base failures had occurred in this surface. Road No. 2 was 6.5 miles long and consisted of a traffic bound aggregate surface approximately 4 in. deep with a normal clay subgrade. The only soil in the aggregate was that which had been forced upward from the subgrade by traffic. Failures had also occurred in this surface. Road No. 3 was 8 miles long and had a 3-in. aggregate surface. There was a normal amount of what is called black "gumbo" clay present in the subgrade. Failures had also occurred in this surface.

Tilley: The experimental project is located on Highway No. 33 beginning at Crete, Nebraska and extending east 4.4 miles. This road carries considerable truck and passenger traffic. The region through which it passes is a glacial area covered with a mantle of loess soils. The soils are primarily silty clays and silty clay loams which have rather high shrinkage factors and are extremely unstable during wet weather. The stabilized base course was designed to provide a 24-ft. surface on the 30-ft. roadway. The first 2.8 miles was to be 5 in. in depth and the next 1.6 miles, 4 in.

DESIGN

Loughborough: Soils and other local surfacing materials are now generally required to meet certain definite specifications, and this has brought about a great improvement in their use and has eliminated many unsatisfactory materials heretofore selected by visual inspection. However, there are certain materials which, failing to meet the general specifications, under certain conditions give good results, and where it is a known fact that such materials have been used satisfactorily and the cost of admixtures of material to bring them up to specifications would be high it would seem only reasonable to set up specifications for those particular materials and allow them to be used. This is of especial significance, because in many States the increasing scarcity of suitable soils is becoming a serious problem. In some localities, soils have been first taken to build a road, then to maintain it for some years as an untreated road, then, finally, to rebuild it. This continuous stripping of top-soil soon depletes the supply and makes all the more important the careful consideration and study of methods whereby the depth of soil necessary to give satisfactory results can be reduced.

Tilley: On the Nebraska project the quantities of coarse sand and selected soil were proportioned so that the final mixture would have about 30 per cent through the No. 200 sieve. The gradations of the materials were such that approximately 60 per cent coarse sand and 40 per cent selected soil were used. The liquid limit of the mix was about 17 and the plastic limit about 12. A density-moisture test run on the material mixed with tar showed that the mix had an "optimum" moisture content of about nine per cent and a maximum dry weight of about 120 lb. per cu. ft. including the tar. Preliminary laboratory tests showed that 4 per cent tar, RT-3 by dry weight, was necessary to stabilize the coarse sand—soil mixture. These tests also indicated that thorough mixing could be accomplished with the moisture content ranging from 8 to 12 per cent and also from 16 to 20 per cent. The material appeared to be practically impossible to mix at moisture contents of from 12 to 16 per cent. The range from 8 per cent to 12 per cent was selected because material mixed at the higher moisture

contents would have required considerable drying before it could have been laid down and compacted.

CONSTRUCTION METHODS

Miller: *Preparation of Windrow—Equipment:* Two Adams motor patrol graders with 12 ft. blades, 1 York Master Workman scarifier and International 40 Tractor, one Killefer Twin-24 in. disk-harrow and one dump body truck part time.

Labor: Two common laborers, four skilled laborers and one grade foreman.

One motor patrol was used the greater part of the time in maintaining the gravel surface and reclaiming and spreading suitable material from the berms and ditches. All material that possibly could be was reclaimed in order that the existing surface would be disturbed as little as possible. All loose material on the surface, together with the reclaimed material, was bladed well onto the shoulders on both sides of the road. The cubical content of the material in the two windrows thus formed was estimated. Additional soil-gravel was loosened from the roadway proper to secure the requisite amount for the 5-in. compacted depth. All the loose material was combined on the shoulders. The exposed subgrade was then dressed to crown and grade and checked with crown board and straight edge. The two windrows were combined to form one large one which was located approximately four ft. off the center line in order to facilitate the movement of traffic. Moisture content, and weight in pounds per cubic foot were determined. The estimated quantity of dry material required to give a 5-in. compacted depth was 1,050 lb. per lineal foot of 18-ft. roadway. From these data the required size of windrow was estimated. A large pair of calipers was used to determine the size of windrow as the contractor was unable to obtain a metering box for a windrow of this size. The cross section of the windrow varied from 9½ to 15 sq. ft. depending on the moisture content, grading and additional material required for widening. Several points of interest may be here emphasized on windrow construction. A York Master Workman was used to loosen the existing gravel surface course. The York machine also had a rake attachment for removing oversize aggregate. Due probably to the large amount of loose material this attachment was practically useless and the oversize material was removed by hand. The disk-harrow was not used often as the fine fraction pulverized very readily when manipulated with the blade. Where an excessive amount of soil fines or a deficiency in gravel existed the contractor was required to carry forward sufficient material to correct this condition. The extra material was obtained by increasing the size of the windrow where an excess of gravel existed, running this material through the traveling mix plant and discharging through the overflow chute into a waiting truck. Whenever a deficiency existed in the windrow additional material was obtained from the shoulder and the Barber-Green overflow. The quantity of material in the windrow was held within 5 per cent of the estimated amount. The lineal feet of windrow prepared was limited to one day's run as the undisturbed material protected the subgrade from drying out and also allowed a quick run-off in case of rain. Approximately 14 hours were required to construct 2,500 and 3,000 ft. of completed windrow with this equipment.

Combining Soil, Gravel, Tar and Water—Equipment: Barber-Green traveling mix plant, water storage tank and pump mounted on a trailer which was attached to the Barber-Green, water pump at the source of supply, two tar distributors—800-gal. capacity each, one 2,600-gal. tar trailer truck, one steam boiler mounted on truck, one 6,000-gal. storage tank and two water trucks.

Labor: Six truck drivers, 2 Barber-Green operators and one common laborer.

Tar Delivery: The tar was heated in the tank car by means of a portable steam boiler. The 2,600-gal. trailer truck transported the tar to the 6,000-gal. storage tank which was located conveniently on the project. The two 800-gal. tar distributors transported the tar from the storage tank to a 700-gal. storage tank on the Barber-Green.

Water Delivery: Three trucks equipped with pumps were required on the first twelve miles of base course construction. The water was supplied by means of two trucks on the remainder of the project by using one large pump at the source of supply and one pump for unloading mounted on the trailer truck.

Mixing: The windrowed material was picked up by the Barber-Green by means of a screw and bucket conveyor which emptied into a storage bin of approximately one cubic yard capacity. The material from the storage bin flowed through an adjustable gate onto a belt conveyor and then into the pug-mill. The amount of material passing through the gate varied with the moisture content and grading of the aggregate. To overcome this discrepancy the windrow was prepared accurately and the gate left full open except where the windrow contained extra material for widening. Where the windrow contained extra material the gate was adjusted to carry the normal amount of aggregate. The forward speed of the plant then was adjusted as required to maintain a constant supply of material in the storage bin. Following this procedure it was possible to change from a windrow of 9.5 to 15 cu. ft. per ft. without stopping the pug-mill.

The tar pump and pug-mill were driven by a motor running at constant speed. The tar was delivered therefore at a constant rate which could be altered only by changing the tar pump gear ratio. The rate at which the tar was delivered and the forward speed of the machine determined the quantity of tar in the finished mixture. Slight variations in the quantity of tar could be obtained by decreasing or increasing the speed of the traveling motor. Major variations could be obtained only by changing the traveling motor gear ratio or both the tar pump and traveling motor gear ratio.

The water was delivered to the pug-mill by means of a pump mounted on the mixer. The quantity of water delivered was controlled manually.

The percentage of tar required in the mix was determined in the laboratory on the soil fines passing the No. 4 sieve having a plastic index of 0 and 15. No allowance was made for bitumen on the coarse material retained on the No. 4 sieve. The average amount of tar required was 5 per cent or approximately 5.0 gal. per lineal foot of windrow. The quantity of water and the gradation of the aggregate were the only two variables that needed to be considered as the rate at which the tar and the soil aggregate were delivered to the mixer was already established. The grading of the aggregate was determined by sampling the windrow where any change in color or texture was noted. This method of sampling was found to be far more effective than obtaining samples at regular intervals. The tar required was calculated for each change in gradation. The inspector at the mixer was then given a tabulation showing the number of gallons of tar required per running foot between designated stations. The travel-motor was usually flexible enough to make a full day's run without changing the gear ratio on the tar pump or traveling motor. Incidentally, it required approximately 30 min. to make a change in the pump sprocket or the gear ratio of the traveling motor. For this reason the rate of tar delivery was set usually at an assumed maximum for the day's

run. The average rate of travel was 5.2 lin. ft. per min. The amount of water required could be estimated closely and was controlled by visual inspection. The moisture content of the mix was carried slightly above the optimum. Keeping the moisture above optimum insured partial evaporation of the light oils when the mix was aired, before spreading and compacting. Moisture content of the windrow during the month of May varied from 1.8 to 6.0 per cent with an average of 2.5 per cent. During the month of August the moisture in the windrow varied from 0.6 to 6.7 with an average of 2.3 per cent. The average amount of water added at the mixer was 5.5 gal. Grade TM-2 tar was used but without much success, as the soil fines were not coated nearly so well as when using TM-1. The maximum length of windrow mixed in one 14-hour day was 4050 ft. The writer was well pleased with the mixing results obtained with the traveling mix plant.

Spreading, Compacting and Finishing—Equipment: One motor patrol with 12-ft. blade, 2 pairs of sheepsfoot rollers, 1 rubber-tired roller, 1 7-ton flat wheeled roller, 1 International rubber-tired tractor and 1 International Trac-Tractor with street plates.

Labor: One foreman, 4 equipment operators and 1 common laborer.

Although the plans call for a uniform cross section 18 ft. wide, it was found advisable shortly after the project was started to increase the width slightly over 18 ft. in order to accomodate a full 18-ft. width wearing surface. The first operation, therefore, before spreading the windrow was to blade all loose material from the roadway. Stakes were then set 10 ft. off the centerline as a guide to the blade operator. A shallow trench was cut 6 in. inside the stake line running from 1 to 1½ in. deep tapering off to zero within 6 in. Spreading and leveling were started as soon as possible. The mixture was then aired to decrease the moisture and light oil content. A tractor-drawn Lakewood rooter was found to be much more efficient than the disk-harrow for this purpose. This operation was followed by using the two sheepsfoot rollers and blading until 1 to 1½ in. of loose material remained on the surface. The sheepsfoot roller was then replaced with the rubber tired roller. Blading and rolling continued until the surface was bound firm and smooth. The flat-wheeler roller gave a final smooth finish to the surface.



Figure 2. Mixing Process About Half Complete. Indiana

Schafer: The construction methods were practically the same for roads No. 2 and 3, and a small part of road No. 1. Stabilization construction was started by scarifying the surface its entire width to a depth of from 4 to 4½ in. This was accomplished by using power graders and a 10 ft. grader equipped with a scarifier attachment and pulled by a 75 H.P. tractor. The power graders next pulverized the material by moving it back and forth over the surface, finally windrowing it at each edge of the proposed base. No lumps larger than ½ in. in diameter were permitted in the mix. The windrowed material consisting of aggregate and subgrade soil was then bladed onto the road bed in three operations, each operation constructing a lift composed of approximately one-third the material. One-third of the total proposed application of tar and enough water to

make the mixture work well was applied to each lift. The total bitumen applied was approximately 0.9 gal. per sq. yd. or 2 per cent by weight of 55-80 (Engler at 40° C) viscosity tar.

After the tar had been applied and all the loose material had been bladed onto the roadbed, the entire mixture was windrowed to the center of the road. At this point a thickened edge was made by cutting a 2 by 36 in. wedge from the subgrade. The material from this wedge was then incorporated with that in the windrow. The entire windrow was then mixed until the tar was uniformly distributed through the mass. Power graders were used for the mixing because the curved blades mix the material in a rolling movement. Motor graders were also used because the entire depth of the material was moved at one time preventing segregation.

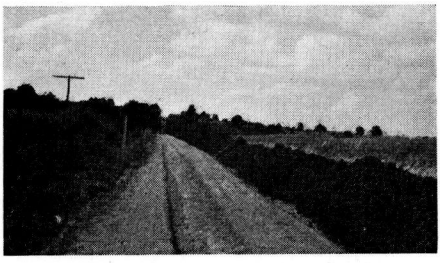


Figure 3. View Showing Thickened Edge Construction of Subgrade

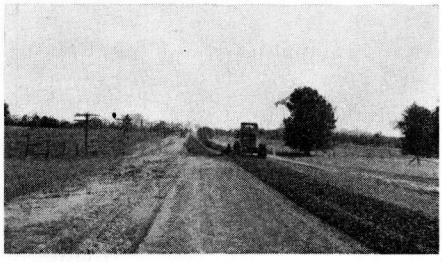


Figure 4. Power Graders Moving Windrows Across the Road, Indiana

After the mixing operation was complete, the materials were again windrowed on each edge of the roadbed. The windrowed material was again evenly bladed onto the roadbed in three equal lifts and each lift compacted by means of rubber tired equipment including a rubber tired roller. A sheepsfoot roller does not work satisfactorily because of the mechanical bond in the mixture. For satisfactory results, kneeding action was required and secured by the use of rubber tired equipment. The last rolling operation was the compaction of a small amount of loose material on the surface with standard 10-ton 3-wheel rollers.

After the mixed materials had been compacted, a tack coat of 0.2 gal. per sq. yd. of the same grade of tar as that used in the mix and 10 lb. of chips per sq. yd. were applied to the surface and rolled. Traffic was permitted to use the surface for ten days and a seal coat was then applied. The seal coat consisted of a 0.2 gal. application of tar and 18 lb. of small chips which were slightly mixed and rolled. To this was applied 0.13 gal. of tar per sq. yd. and enough small chips to get a uniformly tight surface. The surface was then given a final rolling. The tar used for the seal had a float test at 32° C of from 120 to 180 sec.

Road No. 1 was constructed practically the same as roads 2 and 3 except 3.07 miles which were mixed with a traveling plant but spread with motor graders. The bitumen content in the mix of this road varied from 2 to 3 per cent of tar by weight. The average application was 2.5 per cent. This variation in bitumen content was made to take care of the varying amount of fine materials present in the mix.

Moisture tests were not made on any of these projects; however, it was estimated that the total moisture content before mixing was approximately 10 per cent. This deduction was made by estimating the moisture present in the original windrow plus the known amount of water which was added.

On all projects it was necessary to cure the mixture by manipulation which permitted a certain amount of water to leave the mix before it was compacted. Two small areas on one road were not cured sufficiently and these failed. These places were scarified, aerated, again compacted, and sealed. It is estimated that about 4 per cent is the proper moisture content for good compaction.

The sieve analysis of the materials which were in place on the three projects showed a variation of from 18.8 to 91.1 per cent passing the No. 10 screen and from 7.1 to 56.8 per cent passing the No. 200 screen. An examination of the stabilized material showed the particles retained on the No. 10 screen to be practically uncoated while those passing were well coated.

Tilley: The first construction operation to be performed was hauling and placing the coarse sand. This material was obtained from the waste sand pile of an old gravel pit. It was loaded into 4-cu. yd. trucks by means of a $\frac{3}{4}$-cu. yd. dragline and taken an average haul-distance of about five miles. The coarse sand was windrowed on one side of the roadway and the selected soil was placed in a separate windrow.

The selected soil was obtained from local pits adjacent to the project and the average haul distance was a little over one mile. The soil was partially pulverized in the pit by means of farm discs, harrows and tillers and then windrowed with a 12-ft. grader blade. The material was loaded into 4-cu. yd. trucks with two Schinck loaders, a device having a loading arrangement mounted on the front of McCormick-Deering Farmall tractors. The pulverizing was completed on the road by means of the Seaman tiller, a small flat roller and an auto patrol.

The pulverized soil was then placed in a windrow near the center of the road and the coarse sand brought over and placed on the windrow of soil so that both materials formed a single windrow in the center of the roadway. This was the most desirable position from a standpoint of the mixing operation which was to follow and also the grade was better protected in case of rain.

The next operation was to mix the materials in the windrow and apply the tar to the mixed aggregate. This was done by means of a Barber-Greene traveling plant mixer. The mixer took the material from the windrow, mixed it and elevated it into a pugmill at the rear of the machine. The tar and water were added in the pugmill and the mixed material was placed in a windrow at the rear of the mixer ready for the lay-down operations.

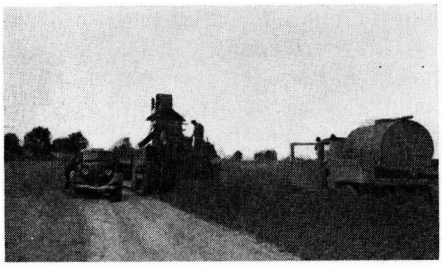


Figure 5. Traveling Plant Discharges Mixed Material in Windrow in Center of Roadbed. Indiana.

The tar was heated to about 145° F. and pumped from tank cars into relay tanks by means of a Cleaver Brooks heater. Two trucks having 1,000-gal. tanks were used to haul the tar to the mixer. These relays hooked on to the mixer and the tar was pumped into a storage tank. The tar was pumped through nozzles into the pugmill by means of a positive displacement pump.

The water was hauled to the mixer with two 1,000-gal. relays and pumped into a trailer storage tank which was hooked on to the mixer. The water was pumped from the storage tank through

a meter and into spray bars mounted over the pugmill, the flow being regulated by means of a hand valve.

The purpose of the water was twofold, first to assist in thorough distribution of the tar and second to aid in compaction. The consistency of the material as it came from the pugmill was kept just below the plastic limit. This could be checked with very little experience by compressing the material between the fingers. The moisture content of the mix was about 9 per cent. The amount of moisture required for compaction was about 9 per cent. When the mix reached a moisture content greater than about 12 per cent, the material became so stiff in the pugmill that mixing was impossible.

The traveling plant mixer was operated 24 hours a day, traveling at the rate of one station in 25 minutes on the 4 in. section and one station in 30 minutes on the 5 in. section.

After a mile of the windrow had been mixed the lay-down operations were started. This required 2 auto patrols, 2 sets of dual drum sheepsfoot rollers pulled by 40 h.p. crawler-type tractors, 1 pneumatic-tired roller and 1 flat roller pulled by rubber-tired farm tractors, and 1 distributor for applying water.

The first operation was to move the windrow from its original position to one side of the road. Next, the base was wetted and material laid on the surface not occupied by the windrow and over to a line of stakes which had been set for the outer edge of the base course. The remaining part of the windrow was moved from its position, the base wetted where the windrow had been, and the material laid to the other stake line. An auto patrol with a shoe on the outer edge of the mold board was used to cut the material back from the outside to form the edge and shoulder line of the base course. The base course was then shaped to the approximate cross-section leaving the material in place but not compacted. It usually took about 3 to 4 hours to reach this stage of the lay-down.

The sheepsfoot rollers, loaded to about 230 lb. per sheepsfoot, were started along one edge and lapped systematically towards the center and the operation continued until the feet were within 2 in. of the surface. Then a patrol was used to maintain a fair size windrow over the surface and keep the top to the desired cross-section while the sheepsfoot rollers continued until they were within about an inch of the top. They were then removed, having been in operation from 2 to 3 hours, depending on the moisture content of the material.

When the sheepsfoot rollers were removed, the auto patrol still had its maintenance windrow but usually by this time the material was getting rather dry and it was necessary to apply water to the surface to keep it in a workable condition. The maintenance windrow was then distributed on the surface as the patrol completed the final shaping. The pneumatic-tired roller, with 45 lb. per sq. in. air pressure in the tires, was then started and it continued rolling for two or three hours until the surface was well compacted. It was necessary at times to use light applications of water during this rolling operation.

The following day a flat roller, with a load of 150 lb. per sq. in., was used to compact the surface. About 5 or 6 hours flat rolling was necessary to compact and maintain the shape of the surface while the moisture content was being reduced to a point where the base course was stable under traffic.

It was found that, if for any reason the moisture content reached a percentage greater than about eleven, the surface could not be finished until the moisture content was reduced to a point where the material was stable. In this case, it was necessary to wet the surface and blade with an auto patrol, finishing with pneumatic and flat rollers.

During the period between the lay-down operations and the application of the prime coat, it was necessary to use light applications of water to aid in reducing the moisture in the base course. For a few days after the lay-down, applications of water made the surface soft and it could be bladed, but after about five days, or when the moisture content in the upper part had dried to 5 or 6 per cent, the applied water did not affect the surface. Also, it was noticeable that after about a week the surface became friable and would dust and ravel.

In about a week's time the base course had dried sufficiently to permit application of the prime coat. RT-3 tar, the same as was used in the mix, was applied at the rate of 0.3 gal. per sq. yd. by means of distributors with 12 ft. spray bars. Traffic was kept off the surface for the first two days after the prime coat was applied.

Usually, on the third day after the prime coat was applied, the surface was in condition for the armor coat. The armor coat consisted of 0.35 gal. of RT-10 tar per sq. yd. and 35 lb. of sand-gravel per sq. yd. The tar heated to a temperature of 250° F. was sprayed on the surface with a distributor. Half of the road was covered at a time. The sand gravel was hauled to the project in dump trucks and applied with a spreader box which was attached to the rear end of the truck. It was necessary for the trucks to back up so that the aggregate was spread ahead of the wheels of both the spreader and the truck. The spreader covered a third of the road at a time so three trips had to be made with the spreader. Application of the tar was regulated so that the tar was never very far ahead of the sand gravel. A drag consisting of a number of wire brushes fastened on to a wooden frame was dragged over the surface to distribute the aggregate evenly. This was followed by a flat roller. It was necessary to maintain the surface of the road with the drag broom and roller for three days after the application of the armor coat.

When the maintenance on the armor coat had been completed, soil was hauled in and the shoulders of the grade were built up level with the top of the finished surface.

COST DATA

Loughborough: The essential data and the cost of a tar base in Virginia constructed during 1939 are:

Length: 2.75 miles.
Width of base: 20 ft. Depth of base: 6 in.
Type of soil: Sand clay with plasticity index of 4.5.
Bitumen used: RT-5.
Quantity: 2.7 gal. per sq. yd.
Seal coat: 0.25 gal. RT-7 and 25 lb. sand per sq. yd.
Wearing course: 0.25 gal. RT-7 and 25 lb. No. 9 chips.
Approximately 2 in. of coarse sand added to road before being stabilized.
Cost:

6 in. base per sq. yd.....	$0.25
Seal and wearing course.	0.12
Sand admixture	0.07
Total cost per sq. yd..	$0.44

Miller: The contract unit price for completed stabilization in West Virginia was 37 cents per sq. yd. or $3900 per mile. Total cost which includes shoulders and ditches and 55 lb. wearing surface was $5700 per mile.

The maximum length of roadway that could be completed in one day (14 hours) was 4000 ft. Approximate time required for the various operations on 1000 ft. of roadway:

	Hr.	Min.
Cutting thickened edge and removing loose material..... ..		20
Spreading and leveling..... ..		60
Aerating		60
Sheepsfooting	3	45
Rubber-tired rolling	2	..
Flat-wheeled rolling	1	..

Schafer: The costs for the three projects in Indiana were as follows:

	No. 1	No. 2	No. 3
Stabilized base per mile (18 ft. wide)	$3,276.72	$1,700.37	$1,874.70
Stabilized base per sq. yd. (18 ft. wide)	0.310	0.161	0.158
Seal per mile (18 ft. wide)	624.11	486.00	1,167.30
Seal per sq. yd. (18 ft. wide)	0.059	0.046	0.099
Total cost per mile (18 ft. wide)	3,900.83	2,186 37	3,042.00
Total cost per sq. yd. (18 ft. wide)	0.369	0.207	0.257

The equipment, labor, and materials costs were as follows:

	No. 1		No. 2		No. 3	
	Cost	% of Total	Cost	% of Total	Cost	% of Total
Equipment						
Per mile (18 ft. wide)	$1,416.42	36	$ 315.63	14	$ 638.72	21
Labor						
Per mile (18 ft. wide)	338 10	9	155.65	7	439.53	14
Material						
Per mile (18 ft. wide)	2,146.31	55	1,715 09	79	1,963 75	65
Total	$3,900.83	100	$2,186 37	100	$3,042.00	100

Road No. 3 received a surface treatment while a light seal coat was applied to roads Nos. 1 and 2. The equipment costs of this project were comparatively high because of insufficient heavy equipment which could scarify the surface. This unbalanced the entire procedure. Heavy rains also caused additional manipulation in the preparation of the materials.

The equipment costs for road No. 1 were high because a traveling plant was used for the mixing. This plant was rented and the costs might have been less if the department had owned the machine. The materials also required much additional manipulation because of heavy rains.

Road No. 2 was an ideal project because the equipment was well balanced and the weather was satisfactory. This project was started on June 14 and the stabilized base work was completed June 27, which is a period of 11 working days. The seal coat was completed July 5.

The equipment used on this project was as follows:

1 75 H.P. crawler tractor.
1 10 ft. blade grader with scarifier attachment.
2 Tandem driven power graders with scarifiers.
2 10-ton 3-wheel rollers.
2 850 gallon bituminous distributors.
1 multiple blade maintainer (for surface seal only).
6 2½ cu. yd. dump body trucks as needed.

Tilley: The contract cost for the 4 in. base course in Nebraska without prime coat or armor coat was approximately $6,300.00 per mile, and for the 5 in., it was approximately $7,600.00 per mile. The prime coat and armor coat cost was approximately $1,700.00 per mile.

The cost of the shouldering is not available at this time since it is in the process of being completed.

The total approximate cost of the tar stabilized soil base course with the wearing course of prime coat and armor coat was $8,000.00 for the 4 in. course and $9,300.00 for the 5 in.

SUMMARY

Schafer:

1. For proper workability approximately 10 per cent of moisture should be

present in the mix. The manipulation costs will be increased materially if the moisture content is too high or too low. Low moisture content requires additional mixing to get proper coating of the particles. High moisture content does not affect the coating of the particles but does require additional manipulation to permit the escape of moisture.

2. The stabilized base must not be sealed with too much moisture in the mixture. It is recommended that two weeks elapse between the tack coat application and the seal coat. This permits the appearance of any failures which are due to excess moisture.

3. If excess moisture failures do appear after the seal coat is applied, the base should be scarified and aerated, again compacted, and sealed.

4. A tack coat must be applied immediately after the compaction of the base is complete and before it is used by traffic. The tack coat penetrates the loose material on the surface and provides a temporary wearing surface. The stabilized base should never be used as a wearing surface.

5. The stabilized base can be used as the first step in stage construction. The application of a seal coat provides a cheap wearing surface; a high type surface may be secured by adding a binder course and a wearing course. This stage construction would permit the improvement of a large number of miles where few miles could be constructed if the entire improvement was made in a single season.

6. The length of the working season can be increased because of the low temperature at which the work can be prosecuted.

7. The cost for this type of improvement is very reasonable because those materials which are already present in the surface and subgrade do not have to be purchased. The only expenditures are for bituminous material, chipping aggregate, labor, and equipment.

8. The stabilized base is composed of practically uncoated large aggregate particles completely surrounded by well coated fine particles. The rubbing or grinding action which the fine particles have on the large particles during the mixing period is a reason for the lack of coating on the large particles.

9. Periodic inspections have been made of the three stabilization projects in Indiana. All have satisfactory riding qualities and their surfaces are in excellent condition. Base failures which occurred before the improvement, have failed to reoccur after the improvement. The cost of this type of construction is reasonable and the results to date are very satisfactory.

Tilley: Since the completed road has been in service only a short time, it is impossible as yet to make any conclusions as to the lasting durability of this type of construction, but it is possible to mention a few items that are worthy of consideration in future designs and construction involving tar stabilized soil as a base course.

Trenching the mat would eliminate the trouble that was experienced in holding and compacting the edges of the mat. In any design, however, provision should be made to protect the edges of the mat since they will become very dry and will break away readily.

The moisture content for mixing should be kept in the range between the "optimum" moisture as determined by the Procter density test and the plastic limit of the mix, preferably nearer the "optimum" moisture content. Higher moisture contents are undesirable from the standpoint of the extra manipulation required to dry the material, because of difficulty in compacting the material, and because of damage that might be done to the subgrade due to the wet material.

The material should be laid down not

later than a day or two after mixing has been completed for best results and ease in handling.

The sheepsfoot rollers should not be brought up closer than about one inch from the surface.

Excess moisture in the lower part of the base course penetrated into the subgrade even after the prime coat was applied.

The completed road has a base course with a density higher than the maximum Procter density; it appears to be impervious to water and the surface has good riding qualities. Therefore, it might be said as a conclusion to the above experience that stabilizing base courses with a small percentage of tar has possibilities that are worthy of further consideration.

PROGRESS IN SOIL-CEMENT CONSTRUCTION

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SYNOPSIS

In this symposium representatives of six State highway departments presented the most interesting features of the use of soil-cement encountered in their practice.

W. H. Mills, Jr., presented the general procedure of soil-cement stabilization as practiced in South Carolina. He reported that if suitable sand-clays or top-soils are available within a 6 to 8 mi. haul, stabilization is not economical. Both road-mix and traveling pugmill methods are used, road-mix methods being preferred in heavy clay soils. Compaction is the most important of the operations, and moisture must be carefully controlled.

Described in detail by L. D. Hicks of North Carolina were methods for sampling and determination of cement requirement in relation to soil classification. Key samples of each "horizon" were used to determine cement required and smaller samples taken from the subgrade at 1000 foot intervals used for field control. The clay soils reacted well with portland cement. The thin, coastal plain sandy top soils generally require a high cement content for successful treatment but they can be economically discarded and the "B" or "C" horizon soils used in the usual manner.

F. W. Vaughan of Mississippi described in detail the field tests, and the control of moisture and compaction on a 24 mi. soil-cement project. Also described is a method of determining cement content after processing. The mixture is digested with hydrochloric acid and ammonium hydrate is added which precipitates iron and aluminum as hydroxides. Calcium in the filtrate is precipitated as calcium oxalate and dissolved in dilute sulphuric acid. Oxalic acid thus released is determined by titration with potassium permanganate and the equivalent weight of calcium is computed. The cement percentage is computed from the calcium-cement ratio.

The Ohio experience reported by R. R. Litehiser and H. E. Brooks is notable for the use of a road building machine which cuts, pulverizes, mixes and hydrates the soil and cement in one operation. Satisfactory relation was found between the moisture content of the mix and that of the compacted roadway. If the moisture in the mixture was within 2 points of the optimum the compaction was within the minimum requirement of the specifications.

The hills of Maryland gave an excellent opportunity, as reported by J. E. Wood, for the exercise of ingenuity in combating the tendency of the material to drift downhill and to lack of traction for the mixing equipment. The experience indicates that rather steep grades and sharp curves can be successfully constructed by the mix-in-place methods.

Carl R. Reid of Oklahoma described the construction of a "cement-modified" subgrade for a concrete pavement including the procedures followed in testing and a tabulation of data on the soils encountered. Cement content varied from 0 to 16 per cent, with an average of 6 per cent. Control sections without cement

and with various compaction methods and degrees of moisture control were also constructed. It is believed that some curing protection should be provided.

E. J. Sampson and H. G. Henderson, also of Oklahoma, reported on a laboratory investigation of the effects of various dispersing agents on the mechanical analysis of soil-cement mixtures. Sodium carbonate, sodium oxalate and sodium silicate were found to be effective in this order. The data also yielded interesting information on the effects of curing soil-cement mixes.

Some cost data are given from South Carolina, North Carolina and Ohio.

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GENERAL PROCEDURE, SOUTH CAROLINA PRACTICE

BY W. H. MILLS, JR.

In South Carolina stabilization is not considered if suitable sand-clays or top-soils are available economically. These materials can usually be hauled six to eight miles before their price in place exceeds that of stabilization.

When no suitable soil can be found within economical haul distance, preliminary samples are obtained for determination of the methods of stabilization and a recommendation for quantities on which bids will be received. If time is available, a complete set of laboratory data is obtained on these preliminary samples before any recommendation is made, if not, a recommendation is based on previous experience in soils having similar gradings and other characteristics. The laboratory tests include mechanical analysis, determination of liquid limit, plasticity index and shrinkage limit and determination of optimum moisture content for compaction on a mixture of soil with the quantity of cement which is estimated will be necessary for stabilization. Artificial weathering tests on specimens of soil-cement mixes containing various percentages of cement are always made. The percentages of cement are varied, depending on the type of soil, but usually the tests are made with 5, 7, and 9 per cent by weight. An estimate is made of the quantity of each type of soil to be encountered in the final grade and the quantity of stabilizer is determined for the average of the project with maximum and minimum limits to care for variations in the soil. Data from these tests are also used in setting quantities for construction by matching similar soils.

During grading operations, soils which require excessively high quantities of stabilizer are eliminated, if economically possible, by placing them in the bottoms of fills or wasting. Generally the soils as found within the limits of the cut sections are used throughout the project but in a few instances select materials from borrow pits are stabilized more economically. For instance, one location on a certain project would have required at least 12 per cent cement for stabilization but by using selected material from a

nearby pit the quantity of cement was reduced to 6 per cent.

After grading operations are completed, check samples from the finished grade must be tested to determine soil changes. The quantities selected for construction work are, of course, based on actual tests of each soil type and most of this information is obtained from the tests on the preliminary samples. However, check tests following the same procedure as mentioned for the preliminary samples are always made on samples obtained after grading operations are complete. The quantity of cement selected for construction is determined from the results of the artificial weathering tests by a comparison of the different specimens in regard to their condition and losses. Usually the quantity selected gives a loss of less than 5 per cent at the end of 20 cycles, but tests are continued to 50 cycles.

MIXING METHODS

Two methods of mixing have been used successfully—the mixed-in-place or road mix method and the traveling mixing plant.

Mixed-in-Place Method: In the road mix method the soil is first pulverized as completely as practicable for the full width and depth. Cement is applied either from bags spaced at specified intervals on the roadway and emptied by hand or by mechanically operated sand spreaders which are attached to dump trucks. A disc harrow set so as to penetrate about half the depth of the pulverized soil should be run over the soil immediately after cement is spread in order to partially cut the dry cement into the soil. This aids materially in reducing losses of cement during the early part of the dry mixing period.

In our work mixing has always been done by heavy road machines supplemented by disc harrows, although special equipments such as orchard cultivators, disc plows, etc., have been used in a few cases to supplement the road machines. Dry mixing is continued until the soil-cement mixture is uniform in color and in order to accomplish this the soil is moved by the blade from the subgrade for the entire width of the road. Cement has been applied to soil containing almost the optimum moisture without resulting in trouble during the mixing period, but mixing is easier if the soil is dry and, of course, the subgrade must be stable.

After dry mixing is completed, samples of the mix are obtained at intervals not greater than 250 ft. and moisture contents are determined by drying. From these data the approximate quantity of water which must be added can be calculated. An adequate allowance of water must be made for evaporation during mixing or delays will occur in adjusting the moisture during the compaction period. The material is spread to the approximate cross-section before the application of water in order to avoid ponding and wet streaks. Water should be applied from a pressure distributor for the entire width in order to obtain uniform distribution and between loads the mix should be disced in order to avoid ponding. Mixing is continued by the use of heavy road machines and disc harrows until the moisture is uniform, and as a check the moisture content is again determined at intervals not greater than 250 ft.

Machine Mixing: The Barber-Green traveling plant increases the rate of construction considerably over the road mix method but figures of relative costs are not available.

The preparation of the soil for this machine consists of scarifying, partially pulverizing, and windrowing at least one day's run ahead. The moisture content of the soil in the windrow is determined and the quantity of water necessary to increase the moisture content to the optimum with an adequate amount for

evaporation is calculated. On the first project cement was applied from a hopper on a platform of the machine so that it went in with the soil at the front of the pugmill. Water was applied from a spray located at intervals above the pug-mill. However, on the last project cement was spread on the windrowed soil ahead of the mixing machine from bags spaced at the correct intervals. This procedure appeared more satisfactory because of the additional mixing action obtained with the loading device and in the hopper and also because it eliminated crowding of trucks and other equipment around the machine. The application of water was also changed to near the end of the pug-mill to increase the length of the dry mixing period.

At first the mix was discharged from the machine into a large windrow and spread by hand to the correct cross-section, but this procedure was altered on the last few projects and motor patrols were used for spreading. In order to perform this work economically it was necessary to accumulate a windrow 300 to 500 ft. long and at times this procedure would leave mixed and wet material in the windrow for as much as 3 to 4 hours. Tests have shown that this does not effect the mix adversely and the resistance to artificial weathering of specimens made at different intervals after mixing does not change appreciably unless the mix is allowed to remain uncompacted for periods greater than six hours.

The mixture obtained with the Barber-Green machine has not always been satisfactory in heavy clay soils. Due to the plasticity of the clay, balls containing a high percentage of moisture and in some instances an excess quantity of cement are frequently discharged from the pug-mill. On most of our projects it has been necessary to supplement the mixing of the Barber-Green with disc harrows and road machines. After the final mixing is completed the material is spread and disced so as to loosen it to the bottom. The moisture content is again determined and adjusted if necessary before compaction begins.

COMPACTION

The Proctor Principle of soil compaction using the sheepsfoot roller in the field was used on the Johnsonville Project constructed in 1935 and has been used on all projects since. It has been observed that the compacting force should be varied with different types of soil in order to secure maximum density. In sandy soils sheepsfoot rollers exerting high unit pressures will not work to the surface but will dig into the mix and keep it loose. In heavy clay soils rollers having low unit pressures will not give satisfactory densities at moisture contents within workable ranges.

The mixture should be shaped to the approximate cross-section all during the compaction period so that uniform compaction can be obtained from the bottom up. Also, it is necessary that the equipment be shifted throughout the width of the road to avoid uneven compaction which may result in streaks much softer and less dense than properly compacted material. More difficulty is always experienced in compacting the edges with present equipment.

The finishing of the surface presents difficulties which have not altogether been overcome. In clay soil it is our practice to continue with the sheepsfoot roller until the feet are running on the surface and penetrating approximately $\frac{1}{8}$ in. The excess material is removed by blading down to the depth of these penetrations. In sandy soils the roller is removed after it has worked to within approximately one inch of the top; the loose material is spread to the correct cross-section and compacted with pneumatic-tired rollers or loaded trucks. It is often necessary to add moisture to the

top inch of the mix during the compacting period due to excess evaporation but such additions must be carefully mixed to avoid scaling. After all blading is completed the surface is again rolled with pneumatic-tired rollers.

Compaction of the mix is one of the essential operations and great care should be exercised during this part of the work. The compaction of the top one-half inch of soil-cement mixtures appears to be one of the weak points in the present construction methods and there have been evidences of softness and scaling for this depth in a few places.

No curing has been specified on any projects constructed to date. So far it has not been demonstrated that this expense is justified. A tar prime is usually applied to the base within 48 hours after construction and in some cases it has been applied the day of construction as soon as the surface dries. A bituminous wearing surface consisting of a single treatment of asphalt and stone or a mixed-in-place treatment with 50 lb. of aggregate and cut-back asphalt is applied before traffic is allowed on the base. The single treatment does not appear to be adequate for ordinary traffic needs.

SAMPLING, SOIL CLASSIFICATION AND CEMENT REQUIREMENT—NORTH CAROLINA

By L. D. Hicks

Four soil-cement base projects were built in North Carolina from 1937 to 1939 comprising totals of 7 miles and 80,984 sq. yd. These bases are considered satisfactory.

A considerable range in soils was covered and the methods of sampling, soil classification and determination of amounts of cement to use are particularly interesting.

SAMPLING PROCEDURE

In order to determine the proper amount of portland cement to use, durability tests must be made of soil-cement mixtures in which varying amounts of cement are used with the particular soil. Since the necessary amounts vary with the kind of soil, it is imperative that the samples truly represent the material to be encountered.

The Bureau of Chemistry and Soils places soils having the same character of profile, the same range in color, structure, consistency, the same sequence of horizons and degree of horizontal development, the same general conditions of relief and drainage and usually a common or similar origin and mode of formation in groups called series. This is the unit of soil classification. Soils from the same horizons of the same series, the A horizon excepted, are very similar. If a sample is taken from the B horizon of a certain series in one locality it will have similar characteristics to another sample taken from the same horizon of the same series in a different locality. It is logical then to conclude that samples taken from all of the horizons of each of the several soil series to be encountered on a project will represent all of the soils on that project. By this same token, it is reasonable to conclude that samples from the same horizons of the same series and taken from different parts of the country will have similar characteristics. The A horizon or topsoil is generally ignored, as soil from this horizon is usually only a few inches thick and rarely occurs alone in the subgrade after the grading work is done. In fact, such a condition should be avoided during grading operations as this horizon generally contains a high percentage of organic matter, which in sandy soils prevents them from reacting with a normal amount of cement.

When a soil stabilization project is contemplated in North Carolina samples are taken of the B and C horizons of each soil series occurring on the project. These samples are called "key" samples and are mixed with various amounts of stabilizing agents—portland cement or bituminous materials—and tested for durability. These samples should weigh at least 200 lb. if portland cement is to be used and 50 lb. if bituminous materials are to be used. When the grading work has been completed, 30 lb. samples are taken from the grade at intervals of 1000 ft. and for the depth to be stabilized. These samples are tested for mechanical analysis, physical test constants, pH, specific gravity, optimum moisture, density; and when portland cement is to be used, compression tests are made on specimens composed of 6 and 10 per cent mixtures of the soil and cement at 2 and 7 days.

TABLE 1

PERCENTAGE OF CEMENT BY VOLUME NECESSARY FOR STABILIZATION

Soil series	No. of tests	B horizon	C horizon
		%	%
Cecil	2	10	6-10
Appling	2	12	10
Durham	2	10	8
Orange	1	10	..
Georgeville	1	12	10
Tirzah	1	10	8
Iredell	1	12	8
Mecklenburg	1	10(B_1), 12(B_2)	..
Orangeburg	1	8	..
Ruston	1	10	10
Norfolk	3	6	..
Bradley	1	10	12
Portsmouth	2	12(A_2)	

Durability tests have been performed, and amounts of cement necessary have been determined for the following North Carolina soils given in Table 1.

Descriptions of the soils in Table 1 are as follows:

Cecil: Derived from acid crystalline rocks such as granites and gneisses. The A horizon is a brownish gray to brownish sandy loam to clay. The B horizon is a stiff, brittle red clay with mica and free quartz.

Appling: Derived from acid crystalline rocks. The A horizon is a yellowish gray sandy loam. The B horizon is a yellowish red or mixed yellow and red gritty clay.

Durham: Derived from acid crystalline rocks. The A horizon is a yellowish gray sandy loam. The B horizon is a yellow friable sandy clay.

Orange: Derived from slates and fine grained volcanic rocks. The A horizon is a gray to light gray silt loam or gravely silt loam. The B horizon is a yellow and gray mottled clay. The soil in this horizon is very plastic.

Georgeville: Derived from slates and fine grained volcanic rocks. The A horizon is a grayish brown to light grayish brown silt loam, gravelly silt loam or slate loam. The B horizon is a red, compact clay to silty clay.

Tirzah: Derived from slates and fine grained volcanic rocks. The A horizon is a dark-brownish red clay. The B horizon is a deep red or maroon-red granular clay.

Iredell: Derived from basic crystalline rocks such as diorite, gabbro, etc. May occur in small deposits due to intrusions of the basic rock in the native rock. The A horizon is a brown to grayish-brown loam or sandy loam. The B horizon is a very plastic dull yellow to greenish-yellow clay.

Mecklenburg: Derived from basic crystalline rocks. The A horizon is a dark brown clay loam. The B horizon is in two layers, B_1 and B_2. The B_1 horizon is red to yellow clay while the B_2 is a plastic yellow clay.

All of the above soils are residual soils and are found in the Piedmont region of

the State. The following are found in the coastal plain region and are derived from sedimentary sands and clays.

Orangeburg: Occurs in areas where the drainage is well established. The A horizon is a gray sand to sandy loam and the B horizon is a friable, bright red sandy clay. This series also occurs as a sand with two layers of A horizon and no B horizon. The A_1 horizon is a brownish-gray sand and the A_2 is a bright red sand.

Ruston: Occurs in areas where the drainage is well established. The A horizon is a gray sand to sandy loam and the B horizon is a friable, yellowish-red sandy clay. This series also occurs as a sand with two layers in the A horizon and no B horizon. The A_1 horizon is a brownish-gray sand and the A_2 is a yellowish-red sand.

Norfolk: Occurs in areas where the drainage is well established. The A horizon is a gray sand to sandy loam and the B horizon is a friable, yellow sandy clay. This series also occurs as a sand with two layers in the A horizon. The A_1 horizon is a gray sand and the A_2 is a pale yellow sand.

Bradley: Occurs in areas where the drainage is well established. The A horizon is a gray sand to sandy loam and the B horizon is a friable, bright red sandy clay to clay. The B horizon is residual Piedmont material which has been covered with sedimentary material. This series is found near the border line between the Piedmont and coastal plain regions.

Portsmouth: Occurs in areas where the drainage is poorly established. The A horizon is a black sand to loamy sand and the B horizon is a gray, sandy clay, frequently mottled with yellow. This series also occurs as a sand with two layers in the A horizon. The A_1 horizon is a black sand and the A_2 is a dull gray sand. This series contains a fairly high percentage of organic matter.

FIELD TESTING

Moisture and density are determined by a laboratory inspector in the field during the processing operations. After the soil has been pulverized, several moisture determinations are made and the number of loads of water necessary to raise the moisture to the optimum plus evaporation plus 1 per cent for the cement are calculated. When the soil, cement and water have been thoroughly mixed, more moisture determinations are made and if found deficient by more than 1 per cent, more water is added; or if found in excess of the optimum by more than 2 per cent, the mixture is allowed to dry out to about the correct amount. Proctor density determinations are made at intervals of 500 ft. after the proper moisture content is reached. At these same points, road density determinations are made when the base has been thoroughly compacted, and reported in terms of the Proctor density test. We find that 95 per cent of the Proctor density is a very good average and insist upon 90 per cent.

Two methods are in use for determining road density; by cores removed from the base by a core drill and by the so-called sand method. Cores cannot be drilled until the base is thoroughly hardened and so the information is obtained too late to improve low densities. Also the densities of the cores are always greater than the freshly compacted base, due to shrinkage. The density of the base prior to hardening is more important because it represents the thoroughness of compaction, a most important factor in the quality of the base.

The sand method consists of cutting a hole in the base, weighing and determining the moisture of the material, and filling the hole with a sand of known density. The road density is the weight in pounds of the material taken from the hole, corrected for moisture at the time of weighing, divided by the volume of the hole in cubic feet.

MOISTURE AND COMPACTION CONTROL, AND DETERMINATION OF CEMENT CONTENT

By F. W. Vaughan

This report is a description of field tests performed and results obtained on a 24-mile soil-cement project located on State Route No. 15 in north-eastern Mississippi. The discussion includes field laboratory control of moisture and compaction and a method for determining the cement content of the soil-cement mixture. Data for each of these tests showing the results obtained are given in Table 7.

Roadway Density Prior to Stabilization: A roadbuilding machine was used

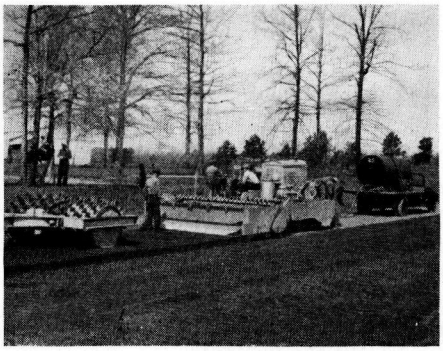


Figure 1. The roadbuilding machine mixing the soil-cement, with the truck tank ahead and the roller behind.

to mix the soil and cement. This machine was designed to mix one-half of the roadway, and mixing was accomplished by lowering a cutting drum to the depth required to yield a six-inch stabilized base. The depth cut by the machine was adjustable and depended on the densities of the raw soil and the completed base.

Prior to processing the soil the roadway was brought to plan grade and cross-section and the density of the material in the top six inches was determined at intervals of 1,000 ft., or less in event of a change in soil type. These density checks were made by drilling to the desired depth with a 4-in. auger. The soil removed was weighed and the moisture content determined. The volume of the hole was found by the sand method.

From the raw soil density and the density anticipated for the compacted processed material, the exact cutting-depth for the mixing machine was determined. The product of the desired depth



Figure 2. Determining the density and depth of the compacted soil-cement base

in inches of compacted material and the ratio of the final desired density (less the cement included in the mixture which equaled 7.5 per cent of the dry weight of the soil-cement mixture) to the density of the raw roadway material determined the depth the machine should cut. See Table 1.

Optimum Moisture and Maximum Density: The soil-cement mixture was designed on average soil samples taken from the roadway prior to the awarding

of the contract. This test work, consisting of moisture-density, wetting and drying, and freezing and thawing tests, was done independently by the department laboratory in Jackson, by the district laboratory in Tupelo, and by the Portland Cement Association. Based on these tests results it was decided to use 7 per cent by dry weight of the mixture which was equivalent to 9 per cent by volume or 0.405 bags per sq. yd. of 6 in. compacted depth. Soon after the construction began, chemical determinations revealed that in some areas the cement content was less than the designed value. To offset this condition the cement content was increased to 7.5 per cent by dry weight of the mixture which was equivalent to 9.56 per cent cement by volume or 0.430 bags per sq. yd. of 6 in. compacted depth.

For more accurate control during construction the grading of the soil was determined at intervals of 200 ft. The groups encountered are given in Table 8. From these gradings it was possible to segregate the soils into more specific types than used in the original design. Representative samples of these various types were submitted to the district laboratory and moisture-density curves of the soil-cement mixtures established. These curves were returned and posted in the field laboratory. Full information was available concerning the soil types to be encountered at each station before the paving operations began.

Field Check on Optimum Moisture During Mixing and Compacting Operations: The amount of water added to the soil-cement was, of course, the difference between the moisture content of the raw soil and that required in the compacted mix. With the data at hand, an inspector was stationed immediately behind the mixing machine to control the amount of water to be added. At frequent intervals samples were taken behind the machine and the percentage of moisture determined in order to insure the proper moisture content. As a result of preliminary laboratory tests on these particular soils an effort was made to maintain the moisture content of the processed material slightly below the optimum. On most soils, moisture should be maintained at the optimum or slightly above. When necessary during the progress of compaction water was added by means of gravity sprinklers mounted on trucks. This additional moisture was required especially during warm weather and when construction delay lengthened the time between the mixing and compacting operations.

Field Check on Optimum Moisture and Maximum Density Curve: In accordance with the variations of soil types additional moisture-density determinations were made on the moist processed

TABLE 1

EXAMPLES OF DETERMINATION OF DEPTH TO BE CUT

Station	Roadway density lb. per cu. ft.	Compacted soil-cement density anticipated lb. per cu. ft.	Depth to be cut in.	Depth obtained in.
777+00	128.0	116.0	$\frac{116-(0.075\times116)}{128}\times 6=5.0$	6.0
785+00	120.0	116.0	$\frac{116-(0.075\times116)}{120}\times 6=5.4$	5.85

material immediately behind the mixer. This was done at frequent intervals and the curves thus established served to keep the optimum moisture and maximum density accurately adjusted. Further, at intervals of approximately 500 ft. or closer if a change in soil type existed, samples of the moist processed material were taken prior to compaction and cylinders molded according to the Proctor Method. Generally these mixtures were molded without the addition of more water; however, in instances where evaporation occurred between the time of sampling and the time of molding, sufficient water was added to produce the designed moisture content. These specimens were properly labeled and sent to the district laboratory for curing and a durability test, which consisted of alternate wetting and drying cycles with brushing at the completion of each cycle. The loss due to brushing measured as a percentage of the dry weight of the specimen was recorded. The freezing and thawing durability test was not performed because earlier investigations of soil-cement mixtures in connection with this project revealed that the wetting and drying test was the more critical for this particular soil.

TABLE 2

EXAMPLE OF MOISTURE CONTROL

Station	Moisture content				
	Raw soil	Optimum soil-cement mixture	Desired	Behind mixer	At completion of compaction
	%	%	%	%	%
225+00	11.6	16	15	14.6	14 5
287+00	11 5	16	15	14.5	14.1

TABLE 3

DURABILITY OF SPECIMENS MOLDED IN THE FIELD LABORATORY

Wetting and Drying Test

Station	Moisture content	Density Proctor cylinder lb. per cu. ft.	Loss from brushing	Cement content		Density of compacted material lb. per cu. ft.
				By weight	By volume	
	%		%	%	%	
854+00	11.9	115.2	13.4	6.6	8.1	118.0
863+00	14.2	113.8	29.3	7.5	9.1	113.8
900+00	14.6	118.0	20.3	6.7	8.4	123.4
972+50	13.7	115.2	7.9	7.5	9.2	118.8
976+00	12.9	117.6	10.3	7.0	8.8	124.9
1061+00	13.6	116.2	11.5	7.2	8.9	116.0

For the purpose of additional information in correlating data, the cement content of each cylinder was determined by an analytical method described later.

Density and Depth Obtained in the Roadway: On completion of the compacting and finishing operations, the density and thickness of the completed work was determined. These checks were made at intervals of 200 ft. and at random points on the cross-section. Since

the joints were compacted by hand, the density here was also determined. The method of checking the density of the completed base was the same as that used in connection with the raw roadway soil. Note that in the representative data in Table 4 the densities are shown at various depths of compaction. It should be noted that the top densities, which are most important, are lower than the densities toward the bottom.

TABLE 4

REPRESENTATIVE DENSITY DATA

Station	Position	Depth in	Density lb. per cu. ft. dry wt.	Theoretical max. density lb. per cu. ft. dry wt	Percentage of theoretical max. density
799+00	Rt.	0–2	115.3	115 0	100.2
		2–4	119.8		104.1
		4–6	119 6		104 0
		Ave.	118 2		102 8
806+00	Rt.	0–2	116.3	116.0	100.2
		2–6	120 2		103 6
		Ave.	118 2		101 9
970+00	Lt.	0–6	115.5	114.0	101 3
	Rt.	0–6	116.1		101.8
1177+00	Lt.	0–3	109.0	114 0	95 6
		3–6	112.4		98 5
		Ave.	110.7		97 0
1204+00	Rt.	0–3	112.7	114.0	98 8
		3–6	119.3		104.6
		Ave.	116 0		101.7

Determination of the Cement Content of the Soil-Cement Mixture: A representative portion (approximately 15 lb.) of the processed material was taken from the roadway behind the mixing machine. A representative sample of raw soil was obtained at the same time and location together with a representative sample of cement. Tests were run on these samples.

First, about 200 g. of the soil-cement mixture are thoroughly dried and pulverized. A 5-g. sample of this is weighed and placed in a 250-cc. erlenmeyer flask, covered with 25 cc. of 1:1 hydrochloric acid solution, and digested on a hot plate for 15 min. After digestion, add water to make 200 cc. (Distilled water is used throughout the procedure.) Allow the solution to boil and set aside to cool. Next add 30 cc. of 1:1 ammonium hydroxide solution and bring again to the boiling point. The ammonium hydroxide neutralizes the excess acid and precipitates the iron and aluminum as hydroxides. Filter hot and wash repeatedly with hot water. Collect the filtrate in a 500-cc. wide-mouth erlenmeyer flash and place on the hot plate. When the solution boils add 25-cc. of a saturated solution of ammonium oxalate. The filtrate thus treated should contain a slight excess of ammonium hydroxide. Calcium will precipitate as calcium oxalate, which should be filtered off and washed with a small quantity of hot water.

Wash the precipitate of calcium oxalate back into the original 500-cc. erlenmeyer flask, add water to make 200 cc., and dissolve with 25 cc. of 1:4 sulphuric

acid. Bring the dissolved oxalate to the boiling point and titrate with a potassium permanganate solution of known normality. An approximately 0.3 N solution is best. From values thus obtained the calcium content may be computed. One cc. of a 0.1 N potassium permanganate solution is equivalent to 0.002004 g. of calcium.

Along with the processed material an analysis is made on a companion sample composed of an accurately weighed 5 g. portion of raw soil to which has been added enough cement to equal 7.5 per cent of the dry weight of the mixture. This is made by mixing 0.375 g. of cement with 4.625 g. of the soil. The soil and cement used are those obtained from construction operations as previously mentioned. This analysis furnishes a factor, the calcium-cement ratio, which is used in computing the grams of cement in the 5-g. sample of processed material analyzed. Determination of this factor is essential due to variations in the calcium content of the soil and of the cement, thus eliminating possible error in assuming a constant factor.

Prior to field laboratory work, the accuracy of this method was determined. Experimental data are tabulated in Table 5.

TABLE 5

No.	Wt. g.	Calcium-cement factor	Calcium g.	Cement g.	Cement recovered %	Cement placed %	Diff.	Efficiency of mixing and sampling %
1	5	0.4363	0 1527	0.3499	7 00	7.00	0 00	100.00
2	5	0 4363	0.1484	0.3401	6.80	7.00	−0.20	97.14
3	5	0.4363	0.1525	0.3494	6.99	7.00	−0.01	99.86
4	5	0.4363	0.1505	0.3449	6.90	7.00	−0.10	98.57
5	5	0.4363	0 1492	0.3420	6.84	7.00	−0.16	97.71
6	5	0.4363	0.1528	0 3502	7.00	7.00	0.00	100.00
7	5	0.4363	0.1475	0.3380	6 76	7.00	−0.24	96.57
8	5	0.4363	0.1293	0.2964	5.93	6 00	−0.07	98.83
9	5	0.4363	0.1483	0.3399	6 80	7.00	−0 20	97.14
10	10	0.4363	0.2954	0.6771	6.77	7.00	−0.23	96 71

NOTE: The average difference between the cement recovered and the cement placed is −0.12 and the average per cent efficiency of mixing and sampling is 98.25.

The above samples were taken from 2500 g. mixtures of sand-clay and portland cement. The soil was taken from the top six inches of the roadway on S. P. 15-1269 A. The factor was obtained by analysis of five gram mixtures containing 0.350 g. of cement and 4.650 g. of soil.

Sample No. 9 represents the primed portion of a cylinder molded according to the Proctor Method and primed with T-C 1 at 140°F.

SUMMARY

Moisture Density Control: In this report, which places emphasis on field laboratory tests for the control of construction operations, it is shown that it is possible to approach closely the desired results. In the first twelve miles of construction, it was found that the average moisture content of the raw soil was 11.6 per cent. The average moisture contained in the mix at the time of compaction was 13.2 per cent as compared to the predetermined optimum of 13.5 per cent. This is excellent moisture control. The average density obtained in the soil-cement base was 113.2 lb. per cu. ft. (dry weight) as compared to an average theoretical maximum of 115 lb. or 98.5 per cent of the theoretical value.

In the second twelve miles the average moisture content of the raw soil was again 11.6 per cent. In this section the

TABLE 6

REPRESENTATIVE CEMENT DETERMINATIONS

Processed material taken behind the mixing machine

Station	Design cement %	Depth of sample in.	Cement recovered %
51+00	7.5	0–1	5.8
		1–6	8.2
95+00	7.5	0–1	7.3
		1–6	7.9
743+00	7.5	0–3	7.5
		3–6	7.9
799+00	7.0	0–2	5.9
		2–3	7 2
		3–6	5.5
817+00	7.0	0–6	6.5
854+00	7.0	0–6	6.6
855+00	7.0	0–6	6.3
863+00	7.0	0–6	7.5
888+00	7.0	0–6	6.9
972+00	7.5	0–6	7.5
976+75	7.5	0–6	7 0
1008+00	7.5	0–6	7.3
1019+00	7.5	0–6	7.2
1027+00	7.5	0–6	6.9
1055+00	7.5	0–6	7.3
1061+00	7.5	0–6	7.2
1066+00	7.5	0–6	7 8
1222+00	7.5	0–6	7.0
1253+00	7.5	0–6	7.6

moisture contained during compaction averaged 14.3 per cent as compared to the predetermined optimum of 15.9 per cent. The data from the first twelve miles show that closer control than this can be obtained. The moisture content obtained on this second twelve miles is lower than is desirable for these mixtures. The average density obtained as a result of field compaction was 114 lb. as compared to the theoretical maximum density of 117 lb. or 97.5 per cent of the theoretical value.

It will be noted that the density control was quite close but varied, as was to be expected, with the accuracy of moisture control.

Determination of Cement Content: The method for determining the cement content of a soil-cement mixture was found to give accurate and informative results. From over 50 tests on soil-cement mixtures of known quantity of cement, the average results obtained were 98 per cent of the known quantity. That is to say, when a mixture containing 7 per cent cement by weight was analyzed, 6.86 per cent was reclaimed.

Since the calcium content varies with different soil types and also varies somewhat with different shipments of cement, for extreme accuracy it is essential that a test be made first to determine the calcium content of each. This, however, is best accomplished by mixing the soil and cement and determining the calcium content of the mixture. This is advisable with each soil type encountered and with each shipment of cement. On this 24-mile project the cement was shipped from one source and the calcium content was found to be fairly uniform. Some variation was found in the calcium content of the soil.

On a four-mile section where 7 per cent cement was specified in the design, 19 samples were tested and were found to contain an average of 6.8 per cent. An 8-mile section on which was specified 7.5 per cent cement, 25 samples were observed to have an average cement content of 7.2 per cent.

TABLE 7

SOIL-CEMENT FIELD TEST DATA

SUMMARY

S. P. 15-1269-A, C and D Chickasaw and Pontotoc Counties, Mississippi

Station	Moisture in raw roadway soil— %	Density of raw roadway soil— lbs. per cu. ft. (dry weight)	Moisture in processed material behind mixer %	Optimum moisture %	Theoretical maximum density— lbs. per cu. ft. (dry weight)	Density of compacted roadway material lbs. per cu. ft. (dry weight)	Depth of compacted roadway material in.	Soil type (See Table 8)
	1	2	3	4	5	6	7	8
27+00	....	.. ..	15.0	16.0	118	111.8	6.25	6
37+00	11.3	112.8	13.2	16.0	119	113.5	6.0	7
55+00	....	....	13 4	14.5	115	113 6	6.0	2
60+80	9.9	123.6	15.2	14.5	115	114.8	6 2	8
70+00	....	.....	12.8	14.5	115	107.9	6.2	8
79+00	....	.....	14.3	16.0	119	111.3	6.2	7
88+00	....	.. ..	15.0	16.0	118	118.5	6.0	6
97+00	....	.....	15.0	14.5	115	116 1	6.0	2
107+00	....	.....	15 0	14.5	115	103 9	5.7	2
117+00	....	.....	13.6	16.0	119	117.6	5.7	7
127+00	....	.. ..	14.5	14.5	115	111.0	5.9	2
137+00	....	.....	16 5	14.5	115	113.2	6.25	2
158+50	10.5	122.9	11 5	16 0	118	116.6	5.9	6
171+00	....	.....	16.5	16.0	119	127.0	5.0	7
181+00	....	.....	14 1	16 0	118	122.8	5 5	6
191+00	....	.....	15.3	16.0	119	110 1	6 0	7
201+00	....	.....	14 1	16.0	118	112 8	6 4	6
211+00	....	.....	13.4	16.0	119	111.7	6.0	7
221+00	11.6	124.8	14.5	16.0	118	113.6	6.4	6
231+00	9.6	127.0	13.5	16.0	118	112.5	7.5	6
255+00	....	.....	12.5	16.0	119	113.0	6.25	7
269+00	....	.. ..	12.0	16 0	119	118.0	6 25	7
287+00	....	.....	14.5	16.0	118	119 6	5.4	6
297+00	....	.....	14.6	16.0	119	115.7	6.75	7
317+00	....	.....	15.5	16.5	114	116.8	6.5	5
327+00	....	.....	13 5	16.0	118	115.1	6 4	6
337+00	10.7	129.1	15 5	16.0	119	114.2	6.4	7
349+00	13.1	128.1	14.2	16.0	118	110.8	7.5	6
359+00	13.4	123.0	14 1	16.5	114	112 1	6.4	5
369+00	....	.....	14.6	16 0	119	119.6	5.5	7
379+00	12.3	128.0	13.6	16.0	118	123.1	5.75	6
389+00	11 1	124 0	12 4	16.0	118	120 2	6.2	6
401+00	....	.....	13 0	16 0	118	111.5	6.25	6
411+00	....	.....	14.5	16.0	118	102 3	6.0	6
433+00	....	.....	13.9	16.0	118	109.1	6 2	6
463+00	....	.....	14.1	16.0	119	109.9	5.7	7
483+00	....	.....	14.6	16.0	118	116.6	4.5	6
503+00	....	.....	13.3	16.0	118	119.0	6.5	6
523+00	11.7	123 8	15 2	16.0	119	118.6	6 25	7
533+00	12.9	126.2	15.5	16.0	119	105.0	6.25	7
555+00	13.4	126.9	14 0	16.0	119	116.1	6.5	7
565+00	12 5	128.5	12.7	16.0	119	113.7	5.75	7
585+00	...	.....	16 0	16 0	118	115.1	5.75	6
595+00	12.7	122.5	13.0	16.0	118	109.7	6.9	6
603+00	12 5	124.2	16 8	16.0	118	121.6	5.9	6
613+00	12.5	124.9	16 0	16 0	118	118.1	5 5	6
622+00	11.3	128.8	15 2	16.5	114	117.3	5.9	5
633+00	....	.....	13.9	16.5	114	108.1	5.75	5

TABLE 7—*Continued*

Station	Moisture in raw roadway soil— %	Density of raw roadway soil— lbs. per cu. ft. (dry weight)	Moisture in processed material behind mixer %	Optimum moisture %	Theoretical maximum density— lbs. per cu. ft. (dry weight)	Density of compacted roadway material lbs. per cu. ft (dry weight)	Depth of compacted roadway material in.	Soil type (See Table 8)
	1	2	3	4	5	6	7	8
643+00	10.6	136.2	11.2	16.0	118	113.7	5.8	6
654+75	10.9	127.4	13.6	16.0	118	111 9	6 8	6
665+00	....	.....	13.0	14.5	115	105.1	5.2	2
685+00	12.4	124.1	13.1	14.0	116	108.8	5.25	1
695+00	12.9	117.1	14.1	14.5	115	103.1	5.75	2
705+00	....	.....	11.9	14.0	116	117 6	6.5	1
715+00	....	.....	12.5	14.0	116	112.4	6 75	1
733+00	.. .	.....	15.6	13.0	114	111.8	7 5	3
743+00	....	.. ..	12 4	13.0	114	108.8	6.75	3
753+00	....	.....	13.2	14.0	116	109.3	6.0	1
763+00	10.3	122.2	14.0	14.5	118	113 9	6.25	4
773+00	10.1	127.1	14.1	13.0	114	119.3	5.2	3
782+00	10.5	127.0	11.1	13.0	114	110.4	7.0	3
792+00	13 0	125.5	15.0	14.0	116	114.7	6.0	1
802+00	10 6	133.4	14.6	13.0	114	112.5	6.2	3
812+00	11.5	115.0	11.9	14 5	115	100.4	6.0	2
841+00	....	... .	12.8	13.0	114	104.5	6.6	3
850+00	....	.....	11.1	13.0	114	111.5	5 6	3
886+20	11.4	125.0	16.0	13.0	114	123.1	5.7	3
894+05	....	.....	14.5	14.5	115	114.3	6.5	2
904+00	....	.....	15 1	14.5	118	112 1	5.9	4
912+00	....	.....	15.2	13.0	114	117.1	6.0	3
939+00	....	.....	13.8	13.0	114	111.8	6 0	3
959+00	....	.....	13.2	13.0	114	121.0	6.1	3
967+00	....	.....	13.7	14.5	118	113.6	6.0	4
978+00	....	.....	12.8	14.0	116	117 5	5.75	1
988+00	....	.....	12.9	13.0	114	114.9	6.25	3
998+00	10.2	128.8	14.0	14.5	118	126.3	6.1	4
1008+00	11 1	120.7	12.1	13.0	114	119.7	6 0	3
1018+00	....	.....	13.5	13.0	114	116.1	5.75	3
1029+00	....	.....	14.4	14.0	116	117.5	6.6	1
1039+00	....	.....	13.0	14 0	116	110.5	6 1	1
1049+00	....	.....	11.6	13.0	114	117.4	5.75	3
1059+00	....	.....	12.5	14.0	116	107 7	6.0	1
1089+00	12.6	123.6	12.1	13.0	114	113.8	5.9	3
1109+90	....	.....	15.3	13.0	114	123.6	6 5	3
1121+00	....	.....	12.7	13.0	114	117.8	6.4	3
1131+00	....	.....	14.1	13.0	114	109 8	5.75	3
1151+00	11.6	126.0	12.9	14.5	118	114.2	6.0	4
1161+00	14.1	122.2	13 1	14.5	118	105.3	5.75	4
1181+00	10 1	119.8	13.5	14.0	116	106.8	6.1	1
1191+00	13.3	118.9	13.3	14.5	118	109.6	6.0	4
1210+00	11.9	118.0	11.7	13.0	114	114.9	6 0	3
1220+00	10.8	124.5	12 4	13.0	114	111.9	6.0	3
1240+00	11.8	123.0	14.5	13.0	114	120.8	6.1	3
1251+00	13.8	120 1	14.0	13.0	114	112.9	6 0	3
1260+00	....	.....	13.0	14.5	118	118.0	6.5	4
1270+00	....	.....	13.2	14.5	118	127.5	5.0	4
1280+00	....	.....	13.0	13.0	114	112.5	6 0	3

Moisture values recorded in column 3 were found from samples taken immediately after the mixing process and prior to compaction. Where these values were appreciably below the optimum (recorded in column 4), more water was generally added by sprinkling the section.

TABLE 8
SOIL GROUPS

Type No.	Retained No. 40 sieve	Coarse sand	Fine sand	Silt	Clay	Soil group and plasticity	Optimum moisture	Theoretical maximum density lb. per cu. ft.
	%	%	%	%	%		%	
1	30–40	65–80	6–12	6–14	10–14	A–1 and 2 F and P	14.0	116
2	25–35	65–80	10–16	8–14	14–25	A–2 VP to A–2 and 7 VP	14.5	115
3	30–40	65–80	6–12	3– 9	0–10	A–3 NP, A–1 to A–1 and 2 F	13.0	114
4	25–35	60–75	14–20	12–18	14–25	A–2 and 7 VP	14.5	118
5	40–50	75–80	8–10	6– 9	0– 5	A–1 to A–3 NP	16.5	114
6	40–45	75–80	7– 9	5– 9	6– 9	A–1 and 2 F and P	16.0	118
7	35–45	70–75	6–12	4– 8	10–13	A–1 and 2 P	16.0	119
8	0–15	35–50	25–45	8–12	10–16	A–1 and 2 P	14.5	115

NOTE: "F" Denotes "Friable"
"P" Denotes "Plastic"
"VP" Denotes "Very plastic"
"NP" Denotes "Non-plastic"

SINGLE MACHINE METHODS AND CONTROL OPERATIONS

BY R. R. LITEHISER AND H. E. BROOKS

The Ohio Department of Highways built its first soil-cement road during the summer of 1939. This road was 6 in. in depth, 18 ft. wide, on a 30-ft. roadway. A bituminous surface treatment was included in the contract. The soil-cement pavement was constructed with 8 per cent cement, except for a length of 1,000 ft. with 10 per cent.

The contractor elected to use a roadbuilding machine for the cutting, pulverizing, mixing, and hydrating of the soil and cement, and instead of the plows, cultivators, harrows and discs usually used as separate units for the mixing operation.

PRELIMINARY LABORATORY INVESTIGATION

A preliminary soil survey was made to determine the different soil types which would be encountered. Durability tests were then run to determine the percentage of cement required for each type of soil. Moisture density tests were also made to determine the maximum dry weight and optimum moisture content of soil-cement mixtures for each soil type. Additional samples were taken at intervals of 500 ft. along the center-line from the top 6 in. of the road, to determine the relocation by the grading operations of the various soil types, so that the correct percentage of cement would be used at each location.

PROCESSING

The roadbuilding machine cut, pulverized, mixed, and applied the water in one operation. The sequence of the processing operations was as follows: Scarifying, blading, spreading the cement, mixing and hydrating the soil and cement with the roadbuilding machine, rolling with sheepsfoot rollers, shaping with a

blade grader, smooth rolling, and curing with a moist straw cover.

The roadbuilding machine consisted of a track-type tractor upon which was mounted a box-like steel platform, the rear end containing two rapidly revolving cylinders to which were spirally attached four rows of hardened steel teeth. As the machine advanced these steel teeth cut into the roadway to a depth of six inches,

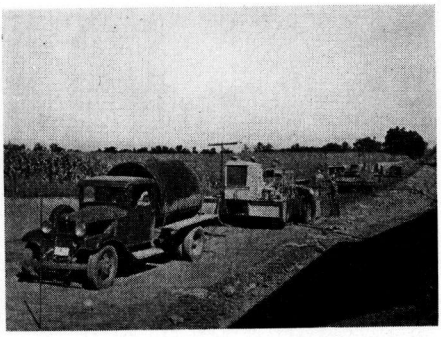


Figure 1. Tank truck supplying water to roadbuilding machine

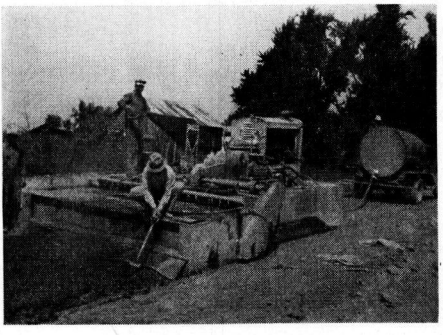


Figure 2. Rear view of roadbuilding machine showing taking of samples of mixture

picked up and pulverized the material, thoroughly mixed it with water and delivered it behind the machine as a level pulverized soil-cement mixture ready for the sheepsfoot roller. The roadbuilding machine processed one-half the width of the pavement at one time. By starting with an initial length of 500 ft., then returning and processing 1,000 ft. on the opposite side, it was possible to continue with staggered sections of 1,000 ft. throughout the day's run. This enabled the finishing operations to be started early and kept about 500 ft. behind the roadbuilding machine.

FIELD LABORATORY CONTROL

A portable field laboratory, mounted on a rebuilt truck chassis, was provided.

Soil samples from the portion of the road to be processed were taken shortly before work started, in order to determine the amount of moisture present and the quantity required to bring the processed soil-cement mixture to optimum moisture content.

With the usual procedure, using cultivators and discs for mixing the soil and cement, and tank trucks for the addition of the water, there is sufficient time to make an intermediate moisture determination, and to calculate the remaining amount of water necessary to obtain the optimum moisture content. However, when using the roadbuilding machine continual attention must be given to the proper addition of water to insure the optimum moisture content, since the very short time required for mixing makes intermediate moisture determinations impossible. It was therefore necessary to devise a method for a quick estimate of the condition of the soil-cement mixture at its optimum moisture content. This was done by vigorously squeezing the moist soil-cement mixture in the hand and noting the consistency. In the field laboratory, moisture determinations on samples of varying moisture content quickly established the proper consistency indicating optimum moisture.

Frequent tests made at the field laboratory for moisture in machine mixed samples, showed the inspector's estimate of optimum moisture content to be very good.

Samples of the machine mix, representative of the day's run, were taken at designated stations so that compaction tests could be made later at the same locations. The moisture content of these samples was determined at the field labo-

ratory and then several density determinations, with varying moisture content, were made on each sample. From these density determinations, partial moisture-density curves were prepared for each sample showing maximum dry weight and optimum moisture content.

Compaction tests of the finished soil-cement pavement were made at intervals of approximately 1,000 ft., at the same stations where the machine mixed samples had been obtained. The compaction tests were usually made the day after laying the pavement since the material was still somewhat soft. To observe the effect of hardening of the soil-cement, other compaction tests were made after seven days' curing and some tests made after two weeks' time. Duplicate tests made at the same locations but after varying lengths of time, showed that difference in the elapsed time before making the compaction tests made no difference in the percentage of compaction.

When the moisture of the soil-cement was 4.4 per cent below optimum, the compaction test on the rolled pavement showed 85 per cent compaction; while with moisture 3.6 per cent below optimum, the compaction test showed 91 per cent compaction. This condition was corrected by the use of more water in the mix and more rolling with the sheepsfoot roller.

Two or more compaction tests were made for each day's processing. Thirty-one were made altogether. All of the tests, after the first day's experimental run, came within the Ohio State highway specification requirement of a minimum of 92 per cent compaction. The 29 compaction tests, omitting the two on the first day's experimental run, gave the following results:

Minimum ..... 92.0 per cent compaction
Maximum ..... 106.0 per cent compaction
Average ..... 99.1 per cent compaction

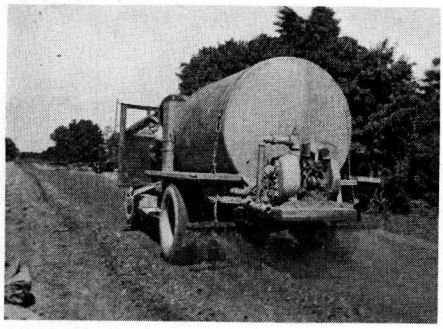


Figure 3. Replacing water lost by evaporation during processing

Some very interesting information was obtained when the percentage figures for moisture in the machine mixes were correlated with the percentages for the corresponding compaction tests. The moisture contents, arranged in groups varying by even percentage from the optimum, gave compaction percentages for each group as shown in Table 1.

TABLE 1

Moisture in machine mix		Number of tests	Compaction, percentage		
Percentage points	Average for group %		Minimum	Average	Maximum
2.0 or more above optimum	+3.7	1	....	97.9	.....
1.0 to 2.0 above optimum	+1.6	4	92.0	98.2	102.2
0 to 1.0 above optimum	+0.1	2	99.3	99.8	100.3
0 to 1.0 below optimum	−0.5	6	96.0	100.8	105.6
1.0 to 2.0 below optimum	−1.5	9	94.8	100.2	104.4
2.0 or more below optimum	−3.6	9	84.6	93.2	97.1
Total tests		31	Av.	98.0	

Table 1 shows that when the moisture in the soil-cement mix was kept within 2 points of the optimum moisture content, the compaction was within the 92 per cent minimum requirement of the Ohio State highway specifications.

The making of the partial moisture density curves, showing the maximum dry weight and optimum moisture content for each of the machine mixed samples, proved to be a great help in obtaining more exact compaction percentages when calculating the compaction tests, than would have been obtained by using the general soil curves for this project, since the maximum density on the same type of soil varied according to the amount of small aggregate present in the sample.

The depth of hardened pavement obtained by measurement at the thirty-one compaction holes showed:

Maximum	6¼ in.
Minimum	5¾ in.
Average	6.01 in.

These results indicate the uniformity of depth obtained on this project with the roadbuilding machine.

OBSERVATIONS ON CONSTRUCTION PROCEDURE

The surface condition of the compacted pavement was influenced by variation, from the optimum, of the moisture content of the soil-cement mix. When either too wet or too dry, rolling cracks developed in the top mulch when it was rolled with the smooth roller. These cracks were transverse to the road; showed during rolling; were not noticeable during the curing period; but could again be seen when the pavement dried out. They were approximately ¾-in. deep.

Where stones larger than one inch were encountered in some areas, and when the finishing operations were delayed, considerable raveling occurred on the road surface after curing and drying. This raveling was not detrimental to this project as the loose stones were brushed off and any depressions were filled by the bituminous surface treatment.

CONSTRUCTION IN THE HILLS OF MARYLAND

By J. E. Wood

The term, "slippery when wet" applies aptly to the stony clay soils of north central Maryland. Add to these features the steep grades and sharp curves incident to the predominantly hilly countryside, and we can realize that here the highway engineer encounters problems that those in more even country escape.

It was under such conditions that a soil-cement road was constructed in Carroll County in August, 1939. The limited appropriation, and the steeply rolling topography, together with the fact that the project was of the farm-to-market class, helped reconcile us to the questionable design of grades up to 10 per cent, and curvature consisting of one 11 deg. 30 min. curve, three 7 deg. curves and one 4 deg. curve.

The surface soil was shallow, being the product of the disintegration of the micaceous shale bedrock lying directly underneath. The soil was still impregnated with fragmentary masses of the rock, varying greatly in size, and ranging 20 to 45 per cent retained on a No. 4 sieve. In places the bedrock was barely beneath the subgrade.

The width processed with soil-cement was 18 ft.; the depth, 6 in. The end furthest from Manchester lay in a valley. From this point, construction followed windingly up-hill to the crest of the ridge, along which the road then extended to the other terminus. Of the seven days' processing, three were spent on the up-hill climb.

Preliminary tests on seven samples

indicated that they could be grouped into two distinct types. Therefore, they were combined to form two composite soils, both classified as A-4 gravelly sandy loams, with liquid limits of 31 and 36, and plasticity indices of 11 and 13, respectively. Moisture-density tests were run on these soils, and specimens molded and subjected to wetting-drying and freezing-thawing durability tests. The data are summarized in Tables 1 and 2.

The laboratory data indicated that the soils could be hardened by the addition of 8 per cent cement by volume, and this was used during construction, except for three sections on which 10 per cent was used. These sections were on grades of 10, 10, and 8 per cent respectively, the first 10 per cent section being on an 11 deg. 30 min. curve superelevated 1½ in. per ft. Details of construction procedures followed on this section will be given later. The third section also included a sharp curve superelevated 1½ in. per ft. The additional 2 per cent of cement used on these three sections was intended as insurance, due to the difficulties of controlling the shifting materials on such steep grades and superelevation. This increase was a comfort at first but as construction proceeded it seemed quite likely that it was not needed as good control was effected in spite of the unusual circumstances.

The methods employed in processing may perhaps be best described by outlining a typical day's run—the second.

On the day previous to processing, the soil bed was prepared; first, by scarifying, and secondly, by pulverizing to a degree so that 80 per cent of the soil would pass through a No. 4 sieve. Owing to the friable nature of the soil and also to the presence of stones, neither scarifying nor pulverizing presented any unusual difficulties. It was found that by scarifying to a depth of about 5½ in. the resulting thickness of the finished soil-cement was approximately the required 6 in.

After pulverization, water was added to bring the moisture content approximately to the optimum. This prewetting had the advantages of:

a—Permitting the moisture, during the night, to equalize throughout the soil bed.

b—Softening any crusted lumps of clay and rendering them more easily broken down.

c—Facilitating the succeeding day's run by shortening the moist mixing period.

d—Doing away with the discomfort of dust and a small cement loss.

e—Lessening the tendency of the dry cement to gravitate to the subgrade where it is hard to recover.

f—Resulting in a more rapid "dry mix" and final mix.

The water was applied to the raw soil with the distributors, each increment being mixed by using a disc harrow and field cultivator. An interesting phase uncovered during the prewetting was demonstrated when the micaceous soil at a moisture content of 9 per cent became so slippery that it provided insufficient traction for the rubber-tired equipment. However, after the cement was incorporated, water could be added to bring the mixture to the optimum moisture content of 16 per cent without being slippery.

Since our typical day's run was on an up-hill grade of 10 per cent and since it extended around a sharp curve calling for maximum superelevation (1½ in. per ft., making the outer edge of the pavement 27 inches higher than the inner, or a cross grade of 12½ per cent), the tendency of the soil with every manipulation to crawl rapidly down grade in both directions, required special attention. It became necessary to operate the motor grader continuously to buffer the soil up-hill and toward the

TABLE 1

GRAIN ANALYSES AND PHYSICAL TEST CONSTANTS

Soil No.	Gradation—Per cent of total							Physical test constants			Specific gravity[2]
	Gravel		Sand		Silt	Clay	Colloids[1]	L. L.	P. I.	S. L.	
	Retained on No. 4 sieve	No. 4 to No. 10 (2.0 mm.)	2.0 to 0.02 mm.	0.25 to 0.05 mm.	0.05 to 0.005 mm.	0.005 to 0.000 mm.	0.001 to 0.000 mm.				
602–A	25[a]	11	16	13	21	12	4	31	11	19	2.70
604–A	20[b]	12	21	16	17	14	9	36	13	22	2.76

[1] Also included in clay fraction.
[2] Fraction passing No. 10 sieve.
[a] Plus No. 4 material: Sp. Gr. = 2.36, absorption = 6.1 per cent.
[b] Plus No. 4 material: Sp. Gr. = 2.21, absorption = 9.2 per cent.

TABLE 2

DURABILITY DATA FROM TWELVE CYCLES OF TESTS

Soil No.	Cement content by volume %		Density lb. per cu. ft. oven dry weight		Moisture content oven dry weight %		Solids by volume, ave. %	Total soil loss % of original dry weight	
	Theoretical	Obtained ave.	Theoretical	Obtained ave.	Theoretical	Obtained ave.		Wet-dry	Freeze-thaw
602–A	0 0*	0 0*	120 6 113.8*	119.9 112.0*	12.8 15.0*	13.6 16.4*	73.8 66.5*	100	100
	6.1 7.5*	6 1 7.5*	118 1 112.1*	118.4 111.7*	12.9 15.0*	13.1 15.2*	72.2 65.5*	3	5
	8.1 10.0*	8.0 9.9*	118 4 111.7*	118.0 110.8*	13.5 15.8*	13.8 16.3*	71.9 64.9*	2	3
	10.2 12.5*	9.8 12.2*	118.8 112.3*	117.2 109.6*	14.1 16 5*	14 4 17.2*	71.4 64 0*	1	3
604–A	6.3 7.5*	6.3 7.5*	112.2 107.6*	112.4 107.6*	16.2 18.0*	16.6 18.4*	68.0 61.8*	Not Molded	4
	8.5 10.0*	8.3 9.9*	111.8 107.2*	111.6 106.4*	16.1 17.8*	16.3 18.1*	67.0 61.2*	2	4
	10.7 12.5*	10.4 12.4*	111.4 106.9*	111.1 105.9*	15.9 17.5*	16.5 18.4*	67.0 60 7*	1	2

* Minus No. 4 material only. Note quantity retained on No. 4 sieve given in Table 1.

high edge of the curve. It was these conditions, together with the extreme stoniness of the soil that limited this day's run to 700 ft.

Processing as such began the following morning, when the first operation was to haul, space and spread the cement.

To guide the spotting of the cement, a 200-foot rope was marked by tags at the correct intervals, the line being moved ahead as the spotting proceeded.

The spotting and spreading required about 1½ hours. (Fig. 1.)



Figure 1. Cement sacks spotted and being dumped to form continuous transverse windrows, preparatory to leveling with spike tooth harrow.

Next came the field cultivator and the disc harrow each of which has its characteristic, yet indispensable, mixing action and each supplementing the other in efficiency. These machines operated about 30 min. or until the free cement had been incorporated with the top soil. The gang plows were then brought into action and operated simultaneously with the other equipment, the plows being of special value in controlling depth of treatment, in cutting the subgrade to a true plane and in bringing up from the bottom any unmixed soil and cement that may have escaped the cultivator and disc. For best efficiency and to avoid turnout, the various units were routed to operate in trains back and forth through the day's run, backtracking in directions with each round trip. (Fig. 2.)

However, on the steep grade it was necessary to work the gang plows in the up-hill direction only, thus also throwing the material toward the high edge of the curve. So great was the displacement of the mix on this hillside run that it became necessary, at times, to stop all other equipment to permit the motor grader to keep the surface in place.

During all mixing operations a gang of men was engaged in casting aside the



Figure 2. Dry mixing operations with two gang plows turning up bottom material, followed by heavy duty field cultivator. Note motor grader in background keeping the continuously shifting, loose material in place.

oversized stones that continually came to the surface.

The application of moisture followed immediately the completion of dry mixing. To minimize the tendency of the water to run down hill, the field cultivator followed the distributor as closely as possible. Since the soil, which had been moistened the night before, now more readily absorbed the added water as it was applied, the moisture mixing was more easily accomplished. The disc harrow was used to a less, and the gang plows to a greater extent than in the dry mixing period. Here, again, the motor grader had the Herculean task of keeping the

surface in shape; but as the soil-cement increased in weight with the added water, the traction of the grader had to be supplemented with a 50 h.p. tractor on the up-hill pull.

When the required moisture had been applied and completely mixed with the soil-cement, the field cultivator was employed to stir up and loosen the entire mass, all other equipment being excluded.

Over this fluffy bed at optimum moisture content the sheepsfoot rollers began to operate. The spike tooth harrow, with teeth set vertically, was attached to one of the rollers to equalize the surface, erase indentations of the roller feet and prevent compaction planes.

When the bottom two-thirds had attained its density, the motor grader was put into service, in a preliminary way, to give the surface its general shape. The rollers continued operating through this stage and, in fact, until the final compaction and final shaping were concluded almost simultaneously.

At this stage, to secure a more uniform surface mulch, about one inch in thickness, the spike-tooth harrow with teeth set vertically and bearing the weight of two men was dragged over the entire compacted area. On the last round-trip with the spike-tooth, the broom drag was attached behind. This device effectively filled in the longitudinal depressions left by the spikes and gave a uniformly distributed loose mulch. (Fig. 3.)

A 10-ton, three-wheeled road roller, beginning at the edges and lap-rolling toward the center, next gave a well knit and dense finish. Better to key in some of the surface stones and erase roller markings, a light spray of water was added and squeegeed in with pneumatic tires. A two-in. earth covering, which was kept moist, was used for cover to prevent evaporation losses.

The stoniness and the slipperiness of the soil and its proneness to shimmy downhill and to the low edge of the curve not only presented problems of interest and even excitement, but also resulted in that degree of satisfaction which only success under difficulties can bring. The surface was excellent, both as to appearance and quality.

In the field, moisture control was governed by tests made:

a—Before applying moisture to the raw soil on the day preceding processing.

b—At the close of the dry mixing period.

c—Near or at the close of moist mixing.

d—After final rolling when density and thickness determinations were made.

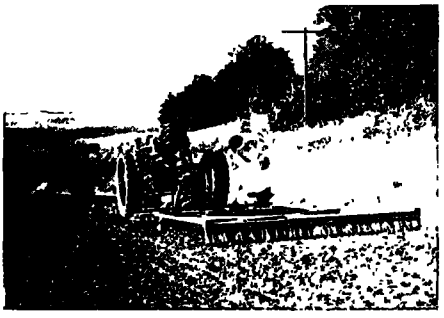


Figure 3. Spike tooth harrow removing compaction planes, followed by broom drag to produce uniformly distributed loose mulch prior to surface consolidation.

A completely equipped mobile soils laboratory [1] gave fingertip control of all the variable elements of the project by making frequent determinations of the No. 4 material, the moisture content and both the density obtained and maximum density.

It was found that with the soil just at optimum moisture content, or slightly above, the free stones in the surface mulch became firmly embedded or healed

[1] See page 296 for a description of the mobile laboratory.

over during smooth rolling. On some of the hot, drying days it became necessary to moisten the surface just before working up the final mulch with the spike-tooth harrow.

During processing, the subgrade was kept under control by constant measurements from a string line stretched between grade stakes. Here close cooperation between the inspector and the operators of the gang plows was essential. During the finishing operations, the same close attention to measurements was given to the final crown and grade in the same way.

A new answer to one detail in soil-cement road construction had been developed in the state of Illinois just before the Maryland work started. This pertains to the joint at the meeting of the current with the preceding day's run. The new method permits the equipment to turn around without driving on or endangering previously completed work. To accomplish this result, the soil for a length of 25 ft. just ahead of the joint was removed entirely to subgrade, and wasted. This space provided ample turning room for the equipment. Near the conclusion of the moist mixing period, a 25-ft. section of the processed soil-cement mixture at the opposite end of the day's run was picked up by a traveling scraper and transferred back to refill

TABLE 3

RESULTS OF DENSITY AND DEPTH DETERMINATIONS

Station	Compacted depth in.	Field density test of minus No. 4 damp mix dry wt., lb. per cu. ft.	Density of minus No. 4 material in completed road, dry wt., lb. per cu. ft.	Density of minus No. 4 material in completed road, % of field density	Density of total material in completed road, dry wt., lb. per cu. ft.
2+00	6 1/16	.....	.....	.....	127.7
4+00	5 5/16	114.0	116.1	101.8	124.8
6+00	5	114.3	116.7	102.1	119.7
8+00	5 3/4	108.3	104.8	96.8	120.7
11+00	5 7/8	111.0	.....	.....	121.5
13+00	5 9/16	.....	.....	.....	121.7
15+00	5 1/2	111.9	111.9	100.0	124.8
17+00	5 5/8	.....	.....	.....	125.2
19+00	5 7/16	110.7	109.6	99.0	117.8
22+00	5 3/8	.....	.....	.....	109.7
24+00	6 1/8	111.0	110.1	99.2	123.8
26+00	5 15/16	111.6	110.4	98.9	119.7
28+00	5 5/8	115.2	118.7	103.0	125.3
30+00	5	.....	.....	.....	118.1
32+50	5 3/4	.....	.....	.....	122.5
35+00	5 1/8	108.6	115.1	106.0	120.0
37+50	6 1/4	.....	.....	.....	117.5
40+00	5 3/4	108.0	113.9	105.5	114.8
42+00	5 9/16	.....	.....	.....	111.3
44+00	5 1/2	.....	.....	.....	117.5
47+00	6	112.2	112.5	100.3	123.3
49+00	5 1/16	111.0	110.1	99.2	121.6
51+00	5 7/8	.....	.....	.....	120.4
54+00	5 1/2	.....	.....	.....	130.2
56+00	5 1/4	.....	.....	.....	130.5

the gap. This transfer was effected just prior to the final turnover of the soil-cement before sheepsfooting.

During construction, control of moisture contents and densities was governed by frequent field moisture-density tests run on the minus No. 4 roadway material toward the conclusion of the damp mixing period. Following completion of each section, density and depth determinations were made at intervals of approximately 200 ft. The results of 25 determinations, ranging from 111 to 131 lb. per cu. ft., including the plus No. 4 material, averaged 121 lb. per cu. ft. for the entire project. The densities of the minus No. 4 material in the completed road, ranging from 97 to 106 per cent, averaged 101 per cent of the values determined by field moisture-density tests at the same locations. These data are tabulated in Table 3.

CONCRETE PAVEMENT SUBGRADE, DESIGN, CONSTRUCTION, CONTROL

By Carl R. Reid

This report, in addition to the records of construction, presents briefly the project design and specifications, the detail of tests for design and control of "cement modified soil" construction, special tests and determinations for comparison of the effect of pulverization of the soil for mixing with cement and the effect of time on moisture-density relations in soil-cement mixtures. Comment is made on construction and suggested methods believed of interest and value in specifying and supervising this type of stabilization.

The project is identified as Federal Aid Project 215-F & G, Comanche County. It is located on U. S. 62 and begins two and one-half miles east of Indiahoma and extends west to the Kiowa County Line, a distance of 7.004 miles.

The original improvement of this highway was completed in October, 1930 to the then standard grade and drainage to a width of 30 ft. and bridges having a roadway width of 22 ft. To complete the improvement in 1938 with high type surfacing required widening the roadbed to 36 ft. and revisions in grade to provide the increased sight distance now required.

Upon completion of the revised plans for this improvement, the usual preliminary soils surveys were made and in addition a study of all local materials was made in order to analyze and report on the probable types or methods of improvement. No examination was made of the density of embankments in place.

Upon examination of the soils analyses it was found that a large portion of the subgrade would be of highly expansive clay soil of the A-6 and A-7 groups, interspersed with less expansive and silty clays of the A-4 group, together with various combinations of all three. It was evident that in order to be assured of a satisfactory improvement, stabilization of a large portion of this subgrade would be required, or a special base and surface design provided to take care of this subgrade condition.

A sample of the heavier clay was sent to the Portland Cement Association, for analyses and recommendations. They suggested that a "cement modified" soil designed for proper reduction of the volume change characteristics should produce a satisfactory, stable subgrade. Their report called for approximately 8 per cent of portland cement by volume to produce this result. Comparing this requirement with soils indicated by the soils survey, estimates for this improvement were prepared which averaged about 6 per cent for the project.

A review of the several types of improvement as to the comparative costs, and the value of finished construction

from standpoint of service, prompted the Oklahoma State Highway Commission, upon the advice and recommendation of Chief Engineer Van T. Moon, to authorize the stabilization of the subgrade of this project with portland cement, to a depth of six in. and to a width of one ft. each side of the proposed 9-in., 6-in., 9-in. by 20 ft. portland cement concrete surface. Construction procedures were to be similar to those used in building light traffic soil-cement roads.

SPECIFICATIONS

Grading and drainage construction was covered by the standard specifications with a special provision that fills of one ft. or more were to be placed in six in. layers, sprinkled as directed by the resident engineer, and rolled. Fills of one ft. or less were not placed in lifts or compacted under the grade and drainage contract.

However, extremely dry weather, under which part of the light fills were placed, made it necessary to sprinkle and compact 6 in. of sub-base directly beneath the proposed soil-cement treatment, throughout these portions. Revised grade line necessitated lowering cuts and raising fills from a minimum of a few inches to a maximum of approximately 4 ft.

"Cement modified" subgrade specifications were based upon those recommended by the Portland Cement Association, as developed and used for base course stabilization, or "cement-hardened soil" projects constructed in other States. The quantity of cement required was to be determined from tests on the soils to be treated, as in place in the top six in. of subgrade upon completion of grading operations. The percentage to be governed by the "volume changes" or "modifications" of the soil and not the "durability" or "hardening" characteristics.

Gradation requirements for soil upon completion of pulverization, were that 80 per cent by weight, exclusive of gravel and stone should pass a No. 4 sieve.

Moisture requirements for the completed blend of cement and soil were that prior to compaction, the moisture content should be within plus or minus 2 per cent of the optimum moisture as determined by the Proctor method.

The compaction requirements were that the uniform completed blend of soil, cement, and water should be compacted to a density equal to 90 per cent of standard dry density as determined by the Proctor method.

FIELD SOILS SURVEY AND DESIGN

In order to avoid any construction delays due to sampling and testing for design of the subgrade treatment, it was found desirable to provide a field soils laboratory.

SAMPLING OF COMPLETED GRADE

Accurate sampling is the first and most important step towards the realization of any comprehensive attempt to analyze soils and correct subgrade, because the value of the most precise and accurate test results can only be judged by the samples they represent. This phase of the soils work presented an unusually difficult problem because of the sharp variations in soil types present in excavations, and these being placed at random in the comparatively light embankment construction. It was very often impossible to assign definite station limits to any particular soil, and in many cases areas were selected having a predominance of a soil type of a more or less uniform combination of several types. After a study of the work as completed and observing grading operations, the method of composite sampling was resorted to, in most instances.

Such representative samples of the soil to be treated were pulverized to pass the No. 4 sieve and divided for the several

portions required by use of a "sample splitter." Duplicate samples in all cases were immediately transmitted to the central laboratory for check test of the field analysis, and also to determine the centrifuge moisture equivalent and mechanical analyses.

The liquid limit, plastic limit, field moisture equivalent, shrinkage limit and shrinkage ratio of the raw soil were determined. From these results the volume ratios were calculated for the equations,

$$\frac{\text{Vol. at S.L.}}{\text{Vol. at F.M.E.}} \times 100$$

and

$$\frac{\text{Vol. at S.L.}}{\text{Vol. at L.L.}} \times 100.$$

The "optimum moisture" was determined by the Proctor Method.

DESIGN

To determine the percentage of cement required in each soil type a series of "trial mixes" with the soil and cement were prepared. Individual 1½-lb. samples of soil, sufficient for about ⅓ or 1½ in. depth in a Proctor mold, were mixed with quantities of cement which were increased in amounts of about 2 per cent to cover the range of cement contents selected to provide specimens having cement contents both above and below the percentage estimated to produce the desired soil characteristics. These trial mixes were prepared at estimated optimum moisture, and compacted in Proctor molds. The specimens were removed from the molds and stored for seven days in the moist closet. At the end of this curing period, they were again ground and the foregoing soil constants again determined. Typical test results of this procedure are shown in Table 1.

TABLE 1

TYPICAL FIELD DESIGN MIX TEST RESULTS

	Raw soil	Soil-cement mixtures			
Cement, by vol. per cent	0	4.0	6.1	8.1	10 1
Cement, by wt. per cent	0	3.5	5.5	7.5	9.5
Liquid limit	45.2	42.3	41 0	39.2	38.6
Plastic limit	21.1	26.2	26 8	28.4	28.3
Plasticity index	24.1	16.1	14.2	10.8	10.3
Field moist. equiv.	28.6	31 6	31 2	32.3	33.7
$\left(\frac{\text{Vol. at S. L.}}{\text{Vol. at F. M. E.}} \times 100\right)$	76.4	83.6	87.4	90.7	89.5
Shrinkage limit	12.7	20 3	22 5	25.9	26.2
Shrinkage ratio	1.95	1.71	1 65	1.59	1.57
$\left(\frac{\text{Vol. at S. L.}}{\text{Vol. at L. L.}} \times 100\right)$	61 1	72.6	76.7	82.8	83 4
Optimum mositure	16 5	....	. .	....	. ..
Moisture of specimen, per cent	.....	19.3	17.3	18.7	17 8
Standard density	109 0	....	....	....	....

Recommended treatment 8.0 per cent cement by volume.

To determine the percentage of cement required in each soil type a series of "trial mixes" with the soil and cement were prepared. Individual 1½-lb. samples of soil, sufficient for about ⅓ or 1½ in. depth in a Proctor mold, were mixed with quantities of cement which were increased in amounts of about 2 per cent to cover the range of cement contents selected to provide specimens having cement contents both above and below the percentage estimated to produce the desired soil characteristics. These trial mixes were prepared at estimated optimum moisture, and compacted in Proctor molds. The specimens were removed from the molds and stored for seven days in the moist closet. At the end of this curing period, they were again ground and the foregoing soil constants again determined. Typical test results of this procedure are shown in Table 1.

The guiding values recommended as being desirable to obtain a subgrade having small volume changes due to moisture changes, were that the "cement modified" subgrade should have a ratio of the volume at shrinkage limit to volume at field moisture equivalent of about 0.90, and a ratio of the volume at shrinkage limit to volume at liquid limit of

about 0.80. The lowest per cent of cement required to obtain these desired results as indicated by the tests in Table 1 was selected as the "Design Mix."

The standard density and optimum moisture of the soil-cement mixture, as thus designed for each soil was determined using the standard Proctor procedure, with exception that a special type hammer, as designed by the Kansas State Highway Department, was substituted. This hammer derives the equivalent of the force of the standard, or original 5.5 lb. Proctor Hammer dropped a distance of 18 in. and is obtained by dropping a 9½-lb. cylinder on a ⅜ in. guide rod. This cylinder is dropped through the controlled height of 12 in. and transfers the energy to a base cylinder having the same area as the standard hammer, and which is attached to the guide rod. There is a loop attached to the upper end of the guide rod which serves as a handle. With this type hammer the operator can readily control the distribution of the required blows over the entire surface of the layer being compacted. Comparative tests indicate that densities obtained with this hammer are slightly higher (102.4 per cent) than those secured by the standard Proctor Test Method, A.A.S.H.O. T-99-38.

FIELD TESTS—RECORDS OF CONSTRUCTION

To determine the compliance with specification requirements for gradation of pulverized soil, composite samples of the material being processed from at least three points across the roadway were taken, at not greater than 300 ft. intervals, or more often as seemed desirable, and in each representative type soil involved. These samples were usually taken as pulverizing was being completed.

As the contractor's work was laid out as to the road length, or stations to be processed, the original mix design requirements were reviewed as to the station limits for quantity of cement required by the construction forces, and the interval, or spacing, per bag of cement furnished the contractor for his use in delivering and spreading the cement.

As dry mixing of the cement and the soil progressed, inspection was made from time to time, to adjust the operation of equipment as necessary to secure a uniform mix throughout the depth and width of the treatment. As dry mixing was being completed, moisture samples were taken throughout the area in much the same manner as samples secured for pulverization tests. From these moisture determinations the amount of water to bring the mix to optimum requirement was estimated. Additional moisture tests were made during damp mixing to assure uniformity, and to ascertain that the optimum requirement had been reached throughout the mixture at which time compaction could be started. These moisture determinations were made over a gas fire, with continual stirring, to secure the earliest possible results.

As the rolling and packing seemed to be completed, density tests were made at selected points in the several types soil, or mixtures, and at intervals not greater than 300 ft. The sand density method was used for this determination, which was compared to the standard density previously obtained for this particular mixture.

At least seven days after processing, four samples were obtained from each days mix for comparative information, for check tests against the original design, and for further tests for information on the effect of aging. In addition, moisture samples were taken just ahead of paving operations, in order to complete records of the project as paving was placed.

CHANGES FOUND NECESSARY—PULVERIZATION

The first section to be processed involved some of the heavier type soils on the project. It was found practically impossible to pulverize this soil to the requirement that 80 per cent pass the No. 4 sieve. Much time was spent with the different pieces of equipment in special methods of manipulation in an effort to secure specification requirements under the cool weather conditions prevailing (late October). The work moved on into somewhat lighter type soils, and when no appreciable improvement in gradation was found, pre-wetting and sprinkling during pulverizing was resorted to. Gradation samples were taken before and after premoistening, during dry mixing with cement, and after final mix was at or near optimum moisture requirement with the result that no appreciable improvement was affected. However, the test data show that additional pulverization was obtained during both dry and damp mixing after cement application. It was quite evident that in these types of soil the equipment, machinery, and methods commonly used for this work could not produce a pulverization of much more than 60 per cent through the No. 4 sieve. It was therefore found desirable and practical to change the specifications as follows:

"That the gradation of the raw soil prior to addition of cement be such that 95 per cent pass the 1-in. sieve, and at least 60 per cent pass the No. 4 sieve."

The average condition, as recorded during the construction of the project, indicates 98 per cent passing the 1-in. sieve, and 67 per cent pasing the No. 4.

That further information might be available as to the effects of pulverization or gradation of the soil on the final blend, soils analyses of completed cement mixtures of both coarse and fine gradings of soil were made, which show no appreciable variation, or difference. Further comparative tests were made on soil-cement cylinders with soil of the coarser or revised pulverization requirement and also of soil, all of which passed the No. 4 sieve. These soil-cement specimens were measured for volume change, and absorption, one each during 26½ days, and one each for 77½ days exposed to capillary moisture, and at the end of that time complete soil analysis was made. None of these results showed any apparent difference to be reflected in either the coarse or the fine graded or pulverized soil. It appears, therefore, that the change to requirement of only 60 per cent passing the No. 4 sieve will have no detrimental effect on this subgrade treatment.

DENSITY

In the early mixes completed it was found that densities in place were not comparable to the (regular) moisture-density curves determined in the laboratory, immediately compacted after mixing the sample. All the methods and procedure were thoroughly checked for errors or discrepancies, with no results. Samples of the mixture in place compacted in Proctor molds showed lower densities indicating that the change or "modification" and moisture-density relation of the soil was effected over a period of several hours during wetting and mixing. A series of moisture-density determinations, delayed for a period of several hours after mixing, developed the fact that the principle change in density was attained in approximately 5 hr.

These "five hr." delayed tests were made during the remainder of the work. The mixture of soil-cement for test was slightly moistened and placed in tight buckets and stirred or agitated at intervals through a period of 5 to 6 hr. after which moisture-density determinations were made using the standard Proctor procedure. Such "five hr." delayed de-

termination, or, comparison of density with compacted sample of the finished road mixture should be specified for control of construction of this type. Project records show the comparison of density and moisture in place with both the "Regular" and "five hr." compaction.

CONCLUSIONS

After carefully determining the requirements for correction of the particular soil in the subgrade, care must be observed to secure the design ratio of this soil to the materials for correction.

Grading must be accurately finished to elevation and section of subgrade for pavement prior to construction of the treated subgrade. Scarifying or "breaking out" this grade to the true section and elevation, at the required depth for the treatment, is of utmost importance. A scarifier, or gang plow, or both, of proper design, and manned by skilled operators should be used throughout, so that this initial work will be carefully controlled and completed to provide a true sub-base section or "work table" for the control of all subsequent pulverization, mixing, processing, and compacting.

The loosening or "breaking out" of six in. of soil in completed roadway provided an excess thickness of finish mix sufficient for any normal irregularities or adjustments in grade or section, and permitted the paving "planer" to cut or trim surplus from the grade rather than to level or fill with untreated or raw soil. As work progressed this surplus was available for the few low areas that occurred in the grade. Proper care and accuracy in this stage of construction is of great importance.

During the initial stages of compaction, no rubber tired equipment or traffic was allowed and the off-set tandem disc was used behind the roller to prevent surface crusting. After about an hour of rolling the disc was replaced with a spiketooth harrow. Particularly during dry weather it was found necessary to sprinkle the surface two or three times during the rolling operation. As rolling neared completion the road was shaped frequently with motor patrol and final compaction was completed using a pneumatic-tired roller.

Contrary to the expectations of some, the final shaping of the completed treated subgrade just before paving was not difficult. The contractor had provided and used, two extra sets of cutting bits for both his subgrade machine and planer, and by changing to new bits as necessary very little delay developed on account of "hard" subgrade. In general, the finished "modified" subgrade was practically perfect as indicated by paving records of the daily yield of concrete, averaging 99.28 per cent, thickness tests and measurements of final cores in the completed concrete pavement.

It is highly desirable that proper and sufficient pieces of equipment for pulverization and processing be available on the work. There is believed to be no substitute for a gang plow (2 to 4 bottom), a regulated depth scarifier, springtooth harrow (power lift and depth regulated), offset tandem disc harrow at least 18- to 20-in. discs, double spiketooth drag harrow, and one way disc plow.

An ample water supply must be available, and it is recommended that application be provided by two or more pressure distributors. On this project, with paving also in progress, a portable water storage tank on the work enabled the distributor trucks to load more quickly than would have been possible from pipe line, and thus deliver water at a satisfactory rate.

All sheepsfoot rollers to be used should be of same general design as to spacing and area of feet and weight.

Results of tests during construction, and the final characteristics of the modified soils, indicate that the revised grading or pulverization requirement of 95 per cent passing 1½-in. and 60 per cent

TABLE 2

EXPERIMENTAL SECTION

Raw Soil—Sta. 1307 + 65 to Sta. 1310 + 65—Field Sample 2

Water-Treated Subgrade—Raw Soil

Pulverization same as cement treated section Sta. 1301 + 50 to Sta. 1307 + 65. Compacted with sheepsfoot rollers.

Moisture and Density (As built 11-10-'38)

Station	Dry den.	Stan. den.	% Stan den.	% Moist.	Opt. moist.
1309 + 25	82.5	96.5	85.5	19.6	26.5

TABLE 3

RESEARCH SECTIONS—CEMENT OMITTED

Station 1016 + 00 to Sta. 1022 + 00—Raw Soil Field Sample 37. Water-Treated Subgrade Section

A. Station 1016 + 00 to Station 1019 + 00—Raw Soil

Water was added to pulverized raw soil as required to provide moisture within 2 per cent of optimum, and mixed in with disc, springtooth harrow and plow to a depth of 6 inches. Packed out with sheepsfoot rollers.

Moisture and Density (As built 12-21-'38)

Station	Dry den.	Stan. den.	% Stan. den.	% Moist.	Opt. moist.
1017 + 50	110.5	115.5	95.7	13.5	14.0

REGULAR PAVING SUBGRADE

B. Station 1019 + 00 to Station 1022 + 00—Raw Soil

Quantity of water added was not predetermined. Rolled with flat wheel roller. Subgrade was not pulverized but was scarified 6 inches deep.

Moisture and Density (As built 12-21-'38)

Station	Dry den.	Stan. den.	% Stan. den.	% Moist.	Opt. moist.
1020 + 50	106.3	115.5	92.0	9.3	14.0

TABLE 4

RESEARCH SECTIONS—CEMENT OMITTED

Station 1120 + 00 to Station 1126 + 00—Raw Soil Field Sample 26. Water-Treated Subgrade Section

A. Station 1120 + 00 to Station 1123 + 00—Raw Soil

Water was added to pulverized raw soil as required to provide moisture within 2 per cent of optimum, and mixed in with disc, springtooth harrow and plow to a depth of 6 inches. Packed out with sheepsfoot roller.

Moisture and Density (As built 11-30-'38)

Station	Dry den.	Stan. den.	% Stan den.	% Moist.	Opt. moist.
1121 + 50	97.8	106.0	92.3	19.8	19.0

REGULAR PAVING SUBGRADE

B. Station 1123 + 00 to Station 1126 + 00—Raw Soil

Quantity of water added was not predetermined. Rolled with sheepsfoot rollers. Section pulverized to a depth of 6 inches before selected for experiment.

Moisture and Density (As built 11-30-'38)

Station	Dry den.	Stan. den.	% Stan. den.	% Moist.	Opt. moist.
1124 + 50	102.6	106.0	96.8	11.8	19.0

Standard Compaction 112.1—Per Cent Standard Compaction 97.9.

TABLE 5

Laboratory Test Results

Station to Station	970+00 976+00		980+40 993+00		993+00 1003+75		1003+75 1016+00		1022+00 1035+00		1035+00 1044+00		1044+00 1068+00		1068+00 1071+00		1071+00 1074+50		1074+50 1080+50		1080+50 1087+50		1087+50 1098+00		1098+00 1110+00		1110+00 1120+00	
Cement by volume, per cent	5.5		5.5		6.0		6.0		6.0		4.0		8.0		7.0		8.0		7.0		8.0		11.0		10.0		9.0	
	Soil[1]	S-C[2]	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C
Sample number	5698	5817	5700	5813	5702	5809	5701	5805	5697	5742	5385	5739	5385	5733	5384	5728	5383	5727	5384	5726	5283	5724	5382	5721	5381	5718	5380	5713
Coarse sand	5.4	7.5	12.0	13.0	19.8	19.5	21.6	23.7	16.5	15.7	13.5	15.8	7.1	7.0	11.6	16.9	7.8	8.8	11.6	13.5	7.8	10.4	7.0	9.6	9.7	10.0	7.0	6.8
Fine sand	28.8	42.1	24.8	40.9	31.4	44.3	32.8	42.7	32.2	48.9	41.0	51.4	23.9	36.2	21.5	36.6	23.3	39.5	21.5	32.0	23.3	42.6	27.5	43.5	28.7	39.4	22.4	51.4
Silt	38.5	49.8	40.6	42.5	30.8	32.6	25.3	30.5	27.1	32.4	28.9	29.8	45.2	54.9	48.0	45.3	47.9	50.0	48.0	51.3	47.9	44.5	45.5	45.4	41.0	49.0	47.4	41.2
Clay	27.3	0.6	23.6	3.6	18.0	3.6	20.3	3.1	24.2	3.0	16.6	3.0	23.8	1.9	18.9	1.2	21.0	1.7	18.9	3.2	21.0	2.5	20.0	1.5	20.6	1.6	23.2	0.6
Colloids	15.2	0.6	12.6	2.6	9.3	2.1	11.2	2.4	13.0	1.4	7.4	1.4	10.3	1.4	8.3	0.7	8.4	1.7	8.3	2.3	8.4	2.5	1.2	1.5	7.3	1.6	9.8	0.6
Liquid limit	38.7	33.3	40.1	36.5	42.0	39.7	43.1	39.9	37.2	33.2	38.3	30.9	48.0	41.5	45.9	39.9	45.5	37.3	45.9	38.1	45.5	41.2	58.3	44.3	49.6	44.9	53.1	43.1
Plasticity index	20.3	9.5	20.8	11.3	25.5	15.6	25.2	11.7	19.7	10.4	19.9	10.4	24.9	13.8	24.6	11.0	22.6	9.2	24.6	10.8	22.6	10.3	32.4	11.8	25.2	14.1	29.6	12.7
Field moist. equiv.	23.8	29.0	24.5	29.2	23.5	32.0	25.5	36.8	21.3	28.2	24.9	25.4	28.9	33.7	27.3	34.2	28.5	31.3	27.3	32.1	28.5	35.4	30.1	37.8	30.2	36.5	28.2	36.3
$\frac{\text{Vol. at S. L.}}{\text{Vol. at F. M. E.}} \times 100$	87.1	94.8	84.8	91.4	85.0	85.2	83.5	89.8	88.2	96.7	84.4	92.8	76.7	87.9	79.5	89.8	77.3	93.5	79.5	88.9	77.3	87.3	72.2	98.2	73.9	86.6	75.4	90.3
Shrinkage limit	15.8	25.3	14.9	24.2	14.1	21.6	14.9	29.1	14.3	26.0	15.0	20.8	13.5	25.0	14.1	26.9	13.6	26.8	14.1	24.3	13.6	26.2	11.4	32.9	12.6	26.7	12.1	28.9
Shrinkage ratio	1.86	1.54	1.88	1.58	1.90	1.67	1.87	1.46	1.90	1.55	1.87	1.69	1.96	1.60	1.96	1.54	1.96	1.54	1.96	1.60	1.96	1.58	2.06	1.45	2.00	1.56	2.01	1.50
$\frac{\text{Vol. at S. L.}}{\text{Vol. at L. L.}} \times 100$	71.7	89.0	67.8	83.7	66.2	76.7	65.5	86.5	69.7	89.8	61.1	85.3	59.8	79.2	61.5	83.3	61.5	85.8	61.5	81.7	61.5	80.8	50.9	85.8	57.5	77.8	54.8	82.4
Soil group	A4-7	A2-4	A4-7	A4-5	A4-7	A4-5	A4-7	A4-5	A4-7	A-4	A-4	A2-4	A6-7	A-5	A6-7	A-5	A6-7	A4-5	A6-7	A4-5	A6-7	A-5	A6-7	A-5	A6-7	A-5	A6-7	A-5
Dry densities[3], lbs. per cu. ft.																												
Regular	113.5		112.5		113.0		115.0		114.5		115.5		111.5		112.0		110.0		112.0		110.0		106.0		109.0		105.0	
5 Hour	106.5		106.0		107.0		109.0		112.0		111.5		104.0		105.0		106.0		105.0		106.0		95.0		103.5		100.0	
Station	971+50		985+00		994+00		1009+00		1025+00		1037+00		1049+00		1068+00		1074+00		1077+00		1083+00		1090+50		1099+50		1111+50	
Roadway	106.4		99.2		106.0		103.7		99.3		101.8		99.3		99.8		93.5		103.5		99.0		99.6		96.2		91.0	
Station	974+50		988+00		997+00		1012+00		1028+00		1040+00		1055+00		1071+00		. .		1080+00		1084+50		1093+50		1102+50		1114+50	
Roadway	109.1		100.7		107.0		106.9		106.6		110.7		97.8		90.8		. .		96.2		99.2		97.8		92.6		86.2	
Station	.		991+00		1000+00		1015+00		1031+00		1043+00		1062+00		. .		....		. .		. . .		1096+50		1108+50		1117+50	
Roadway	. .		97.9		106.5		106.1		100.4		102.2		93.5		. .		..		....		....		92.1		87.7		96.8	

[1] Tests conducted in laboratory on the original raw soil.
[2] Tests conducted in laboratory on the processed soil-cement from the completed subgrade just prior to paving.
[3] Compaction obtained with modified Kansas rammer.

TABLE 5—(*Continued*)

LABORATORY TEST RESULTS

Station to Station	1126+00 to 1144+50		1144+50 to 1148+97		1150+26 to 1151+75		1151+75 to 1158+00		1158+00 to 1162+50		1162+50 to 1181+00		1181+00 to 1189+00		1189+00 to 1197+25		1197+25 to 1207+50		1207+50 to 1212+50		1212+50 to 1219+50		1219+50 to 1222+50		1222+50 to 1236+50		1236+50 to 1246+00	
Cement by volume, per cent	9.5		4.0		4.0		6.0		4.0		7.0		7.0		6.0		4.0		4.0		9.0		4.0		5.5		5.0	
	Soil[1]	S-C[2]	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C
Sample number	5379	5708	5378	5703	5145	5596	5140	5595	5141	5593	5146	5589	5147	5583	5216	5580	5216	5577	5217	5573	5218	5571	5219	5568	5220	5567	5251	5564
Coarse sand	7.7	9.0	9.0	8.7	16.0	16.1	23.5	21.5	13.7	11.4	13.5	17.2	14.4	16.5	20.1	15.4	13.8	18.9	19.4	20.6	11.6	13.9	6.6	7.0	17.0	16.3	18.6	16.5
Fine sand	16.1	39.2	35.1	41.0	31.0	46.6	28.5	47.1	30.6	38.1	18.0	35.4	20.9	40.0	22.5	11.3	28.2	35.4	23.8	30.0	18.2	27.0	20.3	25.4	30.4	31.0	29.2	34.2
Silt	44.1	51.3	40.0	49.8	29.2	36.5	26.3	29.0	33.2	47.7	35.5	46.2	35.8	30.7	34.2	72.1	35.5	44.5	35.1	47.2	39.2	57.0	51.6	63.9	30.4	50.4	26.6	47.2
Clay	32.1	0.5	15.1	0.5	23.8	0.8	21.7	2.4	22.5	2.8	33.0	1.2	28.9	12.8	23.2	1.2	22.5	1.2	21.7	2.2	31.0	2.1	21.5	3.7	22.2	2.3	25.6	2.1
Colloids	11.7	0.5	4.8	0.5	11.8	0.8	13.3	2.4	13.8	1.4	15.9	0.8	11.9	9.7	11.0	0.8	11.0	0.0	11.6	1.3	13.0	1.5	7.9	3.7	9.7	2.3	12.8	1.3
Liquid limit	65.1	50.2	35.2	32.7	40.0	33.0	42.0	35.2	33.0	31.1	50.0	37.1	48.0	44.5	45.5	39.5	32.0	36.7	39.0	34.9	47.5	40.2	33.0	32.4	36.0	35.0	45.0	39.6
Plasticity index	40.8	18.0	17.0	11.9	20.0	8.6	21.5	8.3	14.5	7.2	28.5	11.0	25.5	12.1	21.5	6.6	14.0	12.1	18.5	9.0	27.0	14.0	15.0	8.5	18.0	14.0	26.0	13.2
Field moist. equiv.	27.1	36.4	23.0	23.8	25.0	29.1	26.5	30.4	25.0	28.6	30.5	30.6	28.0	37.1	32.5	36.6	24.5	30.4	27.5	30.5	27.0	33.1	23.5	27.8	24.0	27.3	25.5	31.9
$\frac{\text{Vol. at S. L.}}{\text{Vol. at F. M. E.}} \times 100$	73.7	87.8	88.0	91.8	83.5	91.4	82.8	92.2	85.3	89.0	73.5	94.5	78.1	89.0	75.0	88.8	85.2	89.7	81.7	89.5	76.3	89.4	88.8	95.3	84.0	91.3	80.0	87.1
Shrinkage limit	10.2	27.4	15.8	18.7	14.5	23.4	15.5	25.1	16.0	21.3	12.5	26.8	14.0	29.0	15.0	28.2	15.5	23.6	15.5	3.3	11.5	25.7	16.5	24.6	14.5	21.8	13.0	23.0
Shrinkage ratio	2.12	1.55	1.87	1.75	1.9	1.65	1.9	1.61	1.9	1.67	2.0	1.56	2.0	1.53	1.9	1.50	1.9	1.66	1.9	1.64	2.0	1.60	1.8	1.57	2.0	1.68	2.0	1.66
$\frac{\text{Vol. at S. L.}}{\text{Vol. at L. L.}} \times 100$	46.3	73.8	73.4	80.4	67.3	86.5	65.4	85.9	75.5	85.8	57.1	86.0	59.5	80.8	63.3	85.5	76.0	82.2	69.2	84.0	58.2	81.0	77.2	89.0	69.9	81.8	61.0	78.4
Soil group	A-6	A-5	A2-4	A2-4	A4-7	A-4	A4-7	A-4	A2-4	A2-4	A-7	A4-5	A-7	A-5	A-7	A4-5	A-4	A-4	A4-7	A-4	A-6	A4-5	A-4	A-4	A-4	A-4	A-7	A4-5
Dry densities[3], lbs. per cu. ft.																												
Regular	108.5		115.5		117.0		116.5		116.7		110.5		109.5		110.5		117.5		113.6		111.7		111.6		115.7		114.6	
5 Hour	107.0		111.5		.		.		112.5		101.0		102.5		104.5		111.5		108.0		103.0		110.5		108.5		112.5	
Station	1129+00		1147+00		1151+00		1153+00		1159+00		1163+00		1184+50		1190+00		1198+70		1208+00		1213+00		1221+00		1226+00		1239+50	
Roadway	95.7		109.2		111.4		107.8		101.0		103.7		91.6		98.4		99.9		100.8		103.2		93 2		94.1		106.2	
Station	1135+00		. .		...		1156+00		. .		1166+50		1188+50		1195+40		1204+50		...		1216+00		. .		1229+00		1245+00	
Roadway	100.8		. .		.		100.3		.		90.8		103.4		94.6		99.3		....		102.8		.		105.6		112.6	
Station	1141+00		.		. .		.. .		..		1175+00		....		1196+00		1206+00		....		1219+00		.		1233+00		. .	
Roadway	97.8		....		....		....		...		94.9		...		97.4		106.1		....		94.8		. .		104.8		....	

[1] Tests conducted in laboratory on the original raw soil.
[2] Tests conducted in laboratory on the processed soil-cement from the completed subgrade just prior to paving.
[3] Compaction obtained with modified Kansas rammer.

TABLE 5—(*Continued*)

LABORATORY TEST RESULTS

Station to station	1246+00 1249+00		1249+00 1258+00		1258+00 1261+25		1261+25 1266+50		1266+50 1269+00		1269+00 1272+00		1272+00 1282+00		1282+00 1294+00		1294+00 1301+50		1301+50 1307+65	
Cement by volume, per cent	8.0		8.0		8.0		11.0		8.0		4.0		8.0		8.0		8.0		16.0	
	Soil[1]	S-C[2]	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C	Soil	S-C
Sample number	5253	5560	5252	5558	5253	5556	5254	5554	5253	5553	5321	5552	5322	5551	5323	5547	5324	5542	5135	5540
Coarse sand	12.0	13.9	7.2	10.2	12.0	17.1	8 2	11.1	12.0	16.3	16.2	19.9	18.7	24.9	7.1	10.3	22.9	22.8	2.0	11.6
Fine sand	32.1	41.9	21.1	49.6	32.1	39.0	19.8	41.1	32.1	40.8	40.7	44.3	26.8	54.1	15.4	39.4	19.8	39.5	17.1	45.4
Silt	29.9	39.0	28.7	37.5	29.9	40.7	32.3	45.2	29.9	41.1	29.1	33.5	31.2	17.7	43.0	45.5	31.3	33.6	42.6	38.4
Clay	26.0	5.2	43.0	2.7	26.0	3.2	39.7	2.6	26.0	1.8	14.0	2.3	23.3	3.3	34.5	4.8	26.0	4.1	38.3	4.6
Colloids	12.6	1.3	20.4	1.5	12 6	1.3	22.2	1.5	12.6	0.0	1.7	1.3	11.2	1.3	13.2	2.3	9.6	2.0	14.5	1.6
Liquid limit	49.0	37.6	65.2	40.9	49.0	42.6	58.3	45.5	49.0	42.7	38.8	38.4	49.3	39.1	51.4	42.8	56.3	47.1	88.0	51.1
Plasticity index	28.0	14.0	38.2	11.2	28.0	10.6	35.3	12.3	28.0	15.4	20.8	12.8	28.3	10.7	29.4	12.5	33.5	15.3	50.5	15.0
Field moist. equiv.	26.0	27.6	31.6	34.1	26.0	37.4	26.2	37.5	26.0	33.4	20.4	29.8	26.5	33.7	25.7	35.4	26.1	36.5	49.0	41.4
$\frac{\text{Vol. at S. L.}}{\text{Vol. at F. M. E.}} \times 100$	77.5	91.4	66.9	90.8	77.5	88.9	74.6	95.5	77.5	92.6	91.5	91.5	76.2	95.4	76.6	88.8	77.2	89.8	55.3	91.3
Shrinkage limit	11.5	22.1	9.2	27.8	11.5	29.3	10.0	34.2	11.5	28.3	15.6	24.2	11.3	30.6	10.9	27.4	11.6	29.2	10.5	34.7
Shrinkage ratio	2.0	1.67	2.2	1.58	2.0	1.54	2.1	1.45	2.0	1.57	1.90	1.65	2.05	1.53	2.06	1.57	2.05	1.55	2.1	1.42
$\frac{\text{Vol. at S. L.}}{\text{Vol. at L. L.}} \times 100$	57.1	79.7	42.6	82.8	57.1	83.0	46.3	85.8	57.1	81.5	69.5	84.2	56.0	85.5	54.5	80.4	51.8	78.3	38 2	81.0
Soil group	A6-7	A-4	A-6	A4-5	A6-7	A-5	A-6	A-5	A6-7	A-5	A4-7	A4-5	A-6	A-5	A-6	A-5	A-6	A-5	A6-7	A-5
Dry densities[3], lbs. per cu. ft.																				
Regular	114.0		106.9		102.0		103.5		114.0		120.0		113.5		108.0		107.5		96.5	
5 Hour	112.0		105.9		...		...		112.0		115.5		110.0		105.0		103.5		92.5	
Station	1248+00		1251+00		1259+00		1263+00		1269+00		1272+00		1275+00		1284+50		1295+00		1303+00	
Roadway	103.0		98.2		90.6		81.7		108.0		111.1		104.4		99.8		100.2		84.6	
Station	...		1256+00		...		1266+00		.. .		..		1278+00		1290+50		1298+00		1306+00	
Roadway	.		90.6		.. .		80.0		...		.. .		102.1		97.0		99.4		86.8	
Station	. .		....		...		...		....		....		1278+50		1292+00		1300+00		....	
Roadway	....		....		...		.. .		. .		. .		110.2		98.7		103.2		.. .	

[1] Tests conducted in laboratory on the original raw soil.
[2] Tests conducted in laboratory on the processed soil-cement from the completed subgrade just prior to paving.
[3] Compaction obtained with modified Kansas rammer.

passing No. 4 sieve, should be equal to any soils of finer grading or pulverization, and since practically all of this heavy clay soil (except one stretch of heavy clay with coarse sand) was readily pulverized to this new requirement, it is recommended that such specification be used in future pulverization projects on which "modified soils" are to be produced. Or, that practical grading limits for the soil to be treated be predetermined and wider limits provided if necessary.

It is recommended that density of completed work be specified as the percentage of the density of "five hr." delayed compaction.

No protection or curing was specified for this work. It is believed, however, some provision for protection of cement mix in progress should be provided. On one section heavy rain fell on the finished mix just as rolling started. Subsequent tests indicated the volume change values were not as designed. Two per cent additional cement was reprocessed in a portion of the section, and 5 per cent in the remainder.

RESEARCH SECTIONS

Three sections for future comparison of results were provided in soils of a type requiring treatment. At these locations no cement was added to the raw soil.

One of these sections was brought to optimum moisture and compacted with sheepsfoot rollers (Table 2).

Half of another section was completed as above and the remainder was sprinkled without moisture control, and rolled with the sheepsfoot roller (Table 3).

On the third section, half was compacted at optimum moisture with sheepsfoot rollers. The other half was shaped, sprinkled (no optimum control), and rolled with a 5-ton tandem roller, as provided in the State standard specification for preparation of subgrade for pavement.

DATA

Table 5 summarizes the laboratory test results for all sections of the work.[2]

[2] Complete details of field and laboratory tests for all sections and for each cement used are on file with the Highway Research Board and available on special inquiry.

Additional data on the moisture content of the "modified soil" at the time of placing the concrete pavement and on the hours of rolling required or performed are on file with the Oklahoma State Highway Commission.

DISPERSION OF SOILS AND SOIL-CEMENT MIXES

By E. J. Sampson and H. G. Henderson

The original purpose of this experiment was to determine the difference, if any, in the results obtained in the mechanical analysis (A.A.S.H.O. T-88) of soil-cement mixes using standard sodium silicate as the dispersing agent, and other dispersing agents that might prove more effective in dispersing this type of material.

As the experiment progressed other interesting information became evident, and is included herein.

PROCEDURE

Five soils of known cement requirements for satisfactory modification (4.0, 5.3, 6.1, 10.8, and 13.8 per cent by volume) were selected.

Each soil was ground to pass the No. 4 sieve, mixed with the required amount of cement and water, and placed in a bucket having a tight fitting top. The bucket was shaken vigorously at half-hour intervals over a period of five hours, when the

sample was removed and compacted at the optimum moisture content in the Proctor mold using the Kansas modified compaction hammer.

Two full size Proctor specimens were molded of each soil-cement mix to provide material for analysis at 7, 14, 28, 90-day and 1-year cure. The specimens were immediately removed from the mold, and placed in damp sand to cure. The material for each series of tests was cut from the specimen, leaving the balance intact to continue curing.

All tests were conducted according to A.A.S.H.O. (T-87 to T-94), except that sodium oxalate or sodium carbonate was used as the dispersing agent in the mechanical analyses. These two agents were made up as follows: A saturated solution of sodium oxalate and a 3.7 per cent solution of sodium carbonate. This sodium carbonate solution was selected so that the density of these two agents and the standard sodium silicate would be approximately the same. Except where noted, 20 cc. of dispersing agent were used.

As the work progressed additional possibilities became apparent and the tests were divided into four series, each designed to indicate some characteristic. These series and their individual remarks follow.

Effect of Different Dispersing Agents: At least three analyses were made for each soil and soil-cement mixture using a different dispersing agent in each analysis, sodium silicate, sodium oxalate, and sodium carbonate. Table 1 shows these results on mixture cured 14 days. Three photographs of soil-cement suspensions at the end of 24 hours sedimentation, and their mechanical test results are shown in Figure 1.

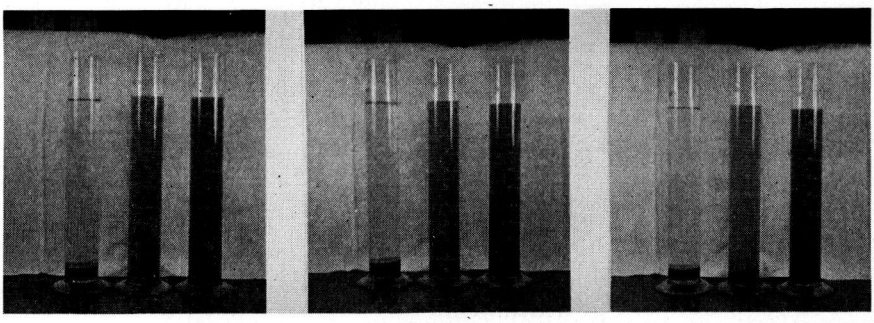


Figure 1. Soil cement suspensions after 24 hour sedimentation. Cured seven days before analysis.

Photo. No.	Cement %	Dispersing agent	Coarse sand %	Fine sand %	Passing No. 200 %	Silt %	Clay %	Colloids %
1	4.0	Sodium silicate....	11.4	59.2	45.6	28.8	0.6	0.6
2	4.0	Sodium oxalate....	16.0	45.8	47.6	30.8	7.4	4.5
3	4.0	Sodium carbonate..	16.0	45.2	48.8	32.4	6.4	1.5
4	5.3	Sodium silicate....	11.6	40.9	54.2	45.6	1.9	1.5
5	5.3	Sodium oxalate....	7.2	48.0	52.6	36.8	8.0	3.2
6	5.3	Sodium carbonate..	9.6	41.0	59.6	40.4	9.0	4.3
7	6.1	Sodium silicate....	15.6	49.4	41.2	33.1	1.9	0.5
8	6.1	Sodium oxalate....	16.2	51.8	40.0	26.1	5.9	2.5
9	6.1	Sodium carbonate..	15.0	51.7	42.2	25.5	7.8	4.5

Variation in Quantity of Dispersing Agent: This series of tests was made on soil-cement mixes cured for 14 days, using double portions (40 cc.) of the dispersing agents. Upon the completion of these tests and a review of the results, another series was run, using either larger or smaller quantities of dispersing agent depending upon the results obtained in the previous tests. This series was intended to furnish preliminary information on the feasibility of conducting future experiments to determine the optimum quantity of various dispersing agents.

Effect of Curing Soil-Cement Mixes: This series of tests was made on material taken from the specimen at the desired curing time. It was intended to use all three dispersing agents on this series, but due to the scarcity of material in some instances, it was impossible to complete the tests using the various agents. For that reason only those tests using sodium silicate are included in this report. See Table 2.

Effect of Prewashing Soil-Cement Mixes: This series was made on material cured 28 days, to permit more complete hydration of the cement, and thus to more nearly simulate the action of water on the modified soil-cement subgrade, and, to indicate the permanency of this transition.

The 50 g. sample for mechanical analysis was stirred in 200 cc. of distilled water, allowed to settle and the clear liquid decanted. This process was repeated for 7 extractions, all extracts being tested for pH values by the colorimetric method. Upon completion of the washing process the mechanical analysis was completed. Triplicate samples were run of each material, using each of the three dispersing agents.

Data: Typical data obtained from these tests, of which there were 145, are given in Tables 1, 2 and 3 and Figure 1.

CONCLUSIONS

Effect of Different Dispersing Agents: These data taken from 75 analyses using 20 cc. of dispersing agent, and 28 analyses using other than 20 cc., a total of 103 including both raw soils and soil-cement mixes, indicate in general an appreciable difference in the effectiveness of the dispersing agents used. The agents in the order of their ability to disperse these raw soils and soil-cement mixes are sodium carbonate, sodium oxalate and sodium silicate.

Visual inspection of the hydrometer tests indicated a slight degree of flocculation in some soil-cement mixes; however the sieve analysis showed in all cases an actual increase in particle diameter or formation of more or less permanent clusters resulting in a coarser gradation than that of the raw soil. See Table 1.

Effect of Variation in Quantity of Dispersing Agent: These data are based upon 45 analyses made using 10 cc. to 60 cc. of dispersing agent. They indicate that variation in the quantity of agent used produces marked variation in results. However, there were no discoverable correlations in the data; for instance, there appeared to be no correlation of the optimum quantities of the different dispersing agents for any particular material; for a given agent, the optimum quantity varied with the particular material. There seemed to be no relationship between the cement content of the soil-cement mix and the optimum quantity of any agent.

The number of analyses made were insufficient to disclose precisely the optimum quantity of dispersing agent in any case; hence the results serve only to indicate the complexity of the problem. Since it is recognized that the stability of a colloidal suspension depends upon both the nature and the concentration of substances present in solution, each soil presents an individual problem in the selec-

TABLE 1

EFFECT OF DIFFERENT DISPERSING AGENTS ON RAW SOILS AND ON 14 DAY CURE SOIL-CEMENT MIXTURES

Field No.	Cement %	Dispersion agent	Coarse sand %	Fine sand %	Passing No. 200 %	Silt %	Clay %	Colloids %
6155–A–I	0	Sodium silicate.....	16.2	39.4	54 0	38.9	5.5	2.1
–A–II	0	Sodium oxalate.....	17.4	35.7	54.2	27.6	19.3	15.0
–A–III	0	Sodium carbonate....	15.2	38.7	54.0	27.8	18.3	14.1
–E–I	4.0	Sodium silicate.....	15.6	57.9	46.6	25.1	1.4	1.1
–E–II	4 0	Sodium oxalate.....	15.8	46.0	46 2	31.8	6.4	3.1
–E–III	4.0	Sodium carbonate...	15.6	46.9	46.0	31.1	6.4	3.1
6158–A–I	0	Sodium silicate.....	10.6	28.6	69.8	45.2	15.6	7.0
–A–II	0	Sodium oxalate.....	9.8	30.2	69.8	36.4	23.6	15.0
–A–III	0	Sodium carbonate....	10.0	27.1	70.4	39.3	23.6	15.2
–E–I	5.3	Sodium silicate	12.0	45.1	54 0	38.6	4.3	1.1
–E–II	5.3	Sodium oxalate.....	9.6	42.5	59.4	40.6	7.3	3.1
–E–III	5.3	Sodium carbonate....	10.2	41.0	58.2	39.4	9.4	6.1
6157–A–I	0	Sodium silicate.....	15.4	38.9	55 6	29 9	15.8	5.0
–A–II	0	Sodium oxalate.....	12.4	36.6	56.6	25.2	25.8	17.2
–A–III	0	Sodium carbonate....	16.2	34 9	55.8	23.1	25.8	17.2
–E–I	6.1	Sodium silicate.....	12.0	57.4	38 8	29.3	1.3	1.1
–E–II	6.1	Sodium oxalate.....	16.8	50.9	39.8	25.0	7.3	5.1
–E–III	6.1	Sodium carbonate ...	16.4	48.5	43.4	25.8	9.3	5.1
6156–A–I	0	Sodium silicate.....	1.2	15.2	97.6	41.7	41.9	22.3
–A–II	0	Sodium oxalate.....	1.2	8 7	97.2	26.2	63.9	4.8
–A–III	0	Sodium carbonate...	1.2	9.4	97.0	26.3	63 1	4.3
–E–I	10.8	Sodium silicate.....	15 0	53 9	35.0	29.6	1.5	1.2
–E–II	10 8	Sodium oxalate.....	12 2	56 2	35.6	30.1	1.5	1.2
–E–III	10 8	Sodium carbonate...	11.2	58 9	34 4	26.6	3 3	3.1
5368–A–I	0	Sodium silicate.....	4 8	13.9	92.8	46 4	34.9	13.1
–A–II	0	Sodium oxalate.....	5.2	10.2	92 4	30.7	53.9	23.4
–A–III	0	Sodium carbonate ...	4.2	10.0	92.8	28.3	57.5	19.3
–E–I	13.8	Sodium silicate... ...	16.8	51.4	36.2	30.3	1.5	1.2
–E–II	13.8	Sodium oxalate.....	17.6	49 3	39.0	31.7	1.4	1.2
–E–III	13.8	Sodium carbonate....	14.2	56.1	35.0	28.2	1.5	1 5

TABLE 2

EFFECT OF CURING ON SOIL-CEMENT MIXTURES DISPERSED WITH STANDARD SODIUM SILICATE

Field No.	Cement %	Curing time days	L. L.	P. L.	P. I.	Shrinkage limit	Ratio	Coarse sand %	Fine sand %	Passing No. 200 %	Silt %	Clay %	Colloids %
6155–A–I	0	..	31.7	18 7	13.0	16 1	1.88	16 2	39.2	54.0	38 9	5.5	2.1
–C–I	4.0	7	34.0	24.2	9.8	22 2	1 71	11.4	59.2	45 6	28.8	0 6	0.6
–H–I	4.0	28	31.3	26.0	5 3	24.2	1.65	16 0	53 1	41.6	29 3	1.6	1.6
–K–I	4 0	90	31 9	28 2	3.7	24.5	1 62	18.4	49.6	39.0	29.7	2.3	0 9
6158–A–I	0	..	33.9	20.5	13.4	16.3	1 85	10.6	28.6	69 8	45 2	15.6	7.0
–C–I	5 3	7	37.0	28.4	8.6	24 2	1 62	11.6	40.9	54 2	45 6	1.9	1 5
–H–I	5 3	28	35.4	29 4	6.0	29.2	1.48	7.8	49 7	48.2	39.0	3.5	2.5
–K–I	5.3	90	36 7	31 2	5.5	27.8	1.54	15.6	44.4	47.6	36 3	3.7	0.9
6157–A–I	0	..	35.1	18.7	16.4	15.0	1.88	15.4	38.9	55.6	29.9	15.8	5.0
–C–I	6 1	7	34 5	28.1	6.4	24.1	1.62	15.6	49.4	41 2	33.1	1 9	0.5
–H–I	6.1	28	34 4	30.2	4.2	27.1	1 54	18.2	54 8	33.2	25.4	1 6	1.6
–K–I	6 1	90	37.4	31 0	6.4	29.8	1.45	24.4	50.5	31.6	23 8	1 3	0.8
6156–A–I	0	..	57.2	27.9	29 3	14.5	1 92	1.2	15.2	97 6	41.7	41.9	22 3
–C–I	10.8	7	49.9	35.8	14.1	28.5	1.57	14 4	52.1	36.6	31.1	2.4	2.3
–H–I	10 8	28	44.5	37 7	6.8	29.6	1.50	22 4	54.4	26 6	21.4	1.8	1 8
–K–I	10.8	90	43.6	40.3	3.3	34 8	1.42	23 6	55.6	25.6	19 6	1.2	0 8
5368–A–I	0	..	87 0	37 8	49.2	10.3	2.13	4.8	13.9	92 8	46.4	34.9	13.1
–C–I	13.8	7	58 6	42.9	15 7	29.4	1.50	20.8	50.4	34.2	26 3	2.5	1.5
–H–I	13.8	28	54.2	45 9	8.3	36 8	1.37	24.6	50.2	27.2	22 7	2.5	1.6
–K–I	13.8	90	52.6	47 4	5.2	44.0	1.27	43.2	39.8	21 0	14 7	2.3	0.9

TABLE 3

EFFECT OF PRE-WASHING 28-DAY CURE SOIL-CEMENT MIXTURES FOR MECHANICAL ANALYSIS

Field No.	Cement %	Dispersing agent	Remarks	Coarse sand %	Fine sand %	Passing No. 200 %	Silt %	Clay %	Colloids %
6155–H–I	4.0	Sodium silicate...	....	16 0	53 1	41.6	29 3	1.6	1.6
6155–J–I	4 0	Sodium silicate...	Washed	12.6	55.9	39.8	28.8	2 7	0.0
6155–H–II	4.0	Sodium oxalate...	....	11 4	54 6	41.2	26 5	7.5	3.5
6155–J–II	4.0	Sodium oxalate...	Washed	16.2	48.9	43 2	28.9	6.0	1.6
6155–H–III	4.0	Sodium carbonate	....	11.6	53.0	43.0	28 7	6 7	3 5
6155–J–III	4.0	Sodium carbonate	Washed	16 0	48.5	45 2	29.5	6 0	3.5
6158–H–I	5.3	Sodium silicate...	....	7.8	49.7	48 2	39.0	3.5	2.5
6158–J–I	5.3	Sodium silicate...	Washed	12.0	51.7	51.6	34.7	1.6	1.6
6158–H–II	5.3	Sodium oxalate...	....	11.8	46.7	50.6	36.0	5.5	3.4
6158–J–II	5.3	Sodium oxalate...	Washed	8.8	51.4	49.8	34.4	5.4	0.9
6158–H–III	5.3	Sodium carbonate	....	12.6	42.5	51.0	36.6	8.3	5 5
6158–J–III	5.3	Sodium carbonate	Washed	11.0	44.5	51.6	37.1	7.4	2.9
6157–H–I	6 1	Sodium silicate...	....	18.2	54.8	33.2	25.4	1.6	1.6
6157–J–I	6.1	Sodium silicate...	Washed	19.0	61.6	32.6	18.9	0.5	0.0
6157–H–II	6.1	Sodium oxalate...	....	20 8	51.6	32.8	22.1	5.5	3.5
6157–J–II	6.1	Sodium oxalate...	Washed	14.0	61.0	31.2	19.6	5.4	2.9
6157–H–III	6 1	Sodium carbonate	....	18.4	54.3	34.6	21.8	5.5	3.5
6157–J–III	6.1	Sodium carbonate	Washed	18.8	54.4	34 2	22.4	4.4	1.9
6156–H–I	10.8	Sodium silicate...	....	22.4	54 4	26.6	21.4	1.8	1.8
6156–J–I	10.8	Sodium silicate...	Washed	26.2	53.3	25.4	18.4	2.1	1.5
6156–H–II	10.8	Sodium oxalate..	....	23.0	57.2	23.4	18.0	1.8	1.6
6156–J–II	10 8	Sodium oxalate...	Washed	20.4	57.1	29.0	20 4	2.1	0.0
6156–H–III	10.8	Sodium carbonate	....	23 8	52.8	27.0	16.0	7.4	3.6
6156–J–III	10.8	Sodium carbonate	Washed	22.4	52.9	28.4	15.3	9.4	3 9
5368–H–I	13.8	Sodium silicate...	....	24.6	50.2	27.2	22.7	2.5	1.6
5368–J–I	13.8	Sodium silicate...	Washed	29.4	49 6	25.4	19.3	1.7	0.6
5368–H–II	13.8	Sodium oxalate...	....	25 4	52.4	26.8	20.5	1.7	0.6
5368–J–II	13.8	Sodium oxalate..	Washed	26.0	51.9	28 8	21.3	0.8	0.0
5368–H–III	13.8	Sodium carbonate	....	27.4	51.0	25.6	19.0	2.6	0.7
5368–J–III	13.8	Sodium carbonate	Washed	28.6	44.0	27.8	24 7	2 7	1.6

tion of the most suitable dispersing agent and its optimum quantity. This fact places the question of optimum quantity beyond the scope of this experiment.

Effect of Curing Soil-Cement Mixes: These conclusions, based on 20 complete tests (see Table 2) can be conveniently enumerated as follows, in general, as the curing time increases:

1. The liquid limit remains practically constant.
2. The plastic limit increases.
3. The plasticity index decreases appreciably.
4. The shrinkage limit increases.
5. The shrinkage ratio decreases.
6. The centrifuge moisture equivalent decreases.
7. The field moisture equivalent increases.
8. The quantity of coarse sand increases.
9. The quantity of fine sand decreases.
10. The percentage passing the No. 200 sieve decreases.
11. The silt decreases.
12. The clay and colloids remain practically constant.

Effect of Pre-washing Soil-Cement Mixtures: Data for this comparison are taken from 30 analyses of 28-day cure material. The salient fact to be noted in this comparison is, that repeated washing did not greatly alter the behavior of the soil-cement mixes during the hydrometer analysis. In samples that showed a tendency to flocculate, pre-washing increased this tendency in some instances, and decreased it in others, but in general, the results were much the same as for the unwashed samples. It is significant that washing did not result in any disintegration of this modified soil, or destroy its alkalinity.

COST DATA

Litchiser, Ohio: The laying of the five miles of soil-cement pavement required 13 working days of approximately 10 hours each. The average length completed per day was 1,827 ft., with record days of 2,200 ft., 2,500 ft., and one day with a maximum of 2,600 ft. A total of 4,782 barrels of portland cement was used on the project, or 956 barrels per mile. Due to the hot dry weather during construction, 600,000 gallons of water were necessary, about one-half being used in the soil-cement mix and the other half being used in the finishing operations and for moistening the straw cover used for curing the pavement.

The cost of the soil-cement pavement and bituminous surface treatment, not including the grading or any other improvements to the road, is as follows:

	Cost per sq. yd.	Cost per mile
Soil-Cement Pavement	$0.38	$3,969
Bituminous Surface Treatment.	.14	1,491
Total	$0.52	$5,460

TABLE 1

SOUTH CAROLINA COSTS

CONSTRUCTED BY CONTRACT

Length miles	Theoretical thickness in.	Cost per sq. yd	Year constructed
10 47	6	$0 495	Winter 1936 Spring 1937
7.7	6	0.480	Fall 1937 Spring 1938
13 7	6	0 50	Summer 1938
4.5	4	0.36	Summer 1938
6.4	4	0.35	Summer 1938
6.8	5	0.35	Summer 1938
0.8	6	0.50	Fall 1939

These prices include application of tar prime.

CONSTRUCTED BY STATE FORCES

1.4	4-6	$0.384	Fall 1935
2.1	6	0.368	Summer 1936

Mills, South Carolina: Prices on several projects are as follows, Table 1.

The costs of projects constructed by the State forces do not include insurance, taxes, etc. Average costs of wearing surfaces were:

Single treatment wearing surface of asphalt and stone approximately 10 cents per sq. yd.

Mixed-in-place treatment with 50 lb. of aggregate and cut-back asphalt approximately 16 cents per sq. yd.

Hicks, North Carolina: Cost data on North Carolina Projects are given in Table 2.

Woods, Maryland: The contract unit prices for the project were: 17 cents per sq. yd. for processing including water requirements and $2.20 per bbl. for cement. Cement contents of 8 and 10 per cent, 0.36 and 0.45 bags per sq. yd. of 6-in. compacted depth were used. The job averaged 0.396 bags or $0.218 per sq. yd. The total average cost for the entire project was $0.388 per sq. yd. or $4,097 per mile.

TABLE 2

NORTH CAROLINA COST DATA

County	Length mi.	Area sq. yd.	Cement used % by vol.	Av. cement used % by vol.	Unit costs per sq. yd.[1]		
					Cement	Hauling and processing	Total S-C base
Carteret	3.1	11,807	11.8	11 8	$0 30	$0.16	$0.46
Beaufort	5 0	15,924	7 to 12	10.39	0.26	0.12	0 38
Wake	2.1	24,582	8 to 12	9 91	0.27	0.14	0.41
Alamance	2 4	28,671	10 to 14	11.51	0.29	0.135	0.425[2]

[1] Built with State forces. Includes rental on equipment and cost of field testing.

[2] Convict labor used.

PROGRESS IN USE OF SODIUM CHLORIDE IN ROAD STABILIZATION

By B. R. Downey, *Maintenance Engineer, Michigan State Highway Department*
R. M. Rowat, *Canadian Industries, Ltd.*
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Arranged by R. W. Crum

SYNOPSIS

An interesting trend in the use of salt for stabilization in Canada was reported by Mr. Rowat. In 1936, the ratio of the quantity of salt used for surface treatment to that used for base stabilization was 4 to 1. In 1939 the ratio had changed to 1 to 14, and the total consumption had increased 15 times in the same period. The central mixing plant was the method preferred for preparation of the material. Salt was used at about 25 lbs. per cu. yd., and costs were about $0.54 per cu. yd. including all cost items except gravel costs.

Mr. Ettinger reports that a cost of $70 per mi. for salt for the first year is to be expected with cost of half that for subsequent years.

Mr. Downey describes the experiments of the Maintenance Division of the Michigan State Highway Department with a type of shallow surface stabilization using a simple road mix procedure.

Rowat: The report of the Subcommittee on Treatment with Sodium Chloride of the Highway Research Board which was published early in 1936 as Part 5 of the Second Progress Report of the Project Committee on Stabilized Soil Road Surfaces provided answers to most of the questions implied in the single one "Will it Work?" It covered the how and the how much, the why, the where and the when.

The economic angle to the use of salt for road-stabilization was not covered by this report.

The 1936 report, however, showed that the design essentials for stabilized soil mixtures when using sodium chloride are similar to those with other chemical admixtures for soil densification, in that design depends upon the proper proportioning of soils, aggregates and admixture. There have to my knowledge been no developments since the publication of that report to alter this conclusion. The modifications in design of stabilized soil mixtures which follow from the purpose for which the materials are to be used, i. e.—base or surface course—apply with equal force when salt is used as when some other chemical is employed.

The use of salt for road construction purposes in Canada has grown steadily since 1934.

One very interesting fact in this growth is that the purposes for which salt is used are changing. Originally, salt stabilization was used entirely in the construction of low cost secondary surfaces. In this use the high degree of compaction secured in the presence of salt, the retention of this compaction under changing external conditions and the increased resistance to frost damage, in some cases amounting almost to immunity, drew attention to the probable value of salt as a stabilizing agent for base courses under permanent or semi-permanent tops, and during the last three years salt base-stabilization has become an important practice.

As an indication of the trend the following may be interesting. In 1936, the ratio of salt consumed in surface stabilization in Canada to that used in base stabilization was 4 to 1. In 1939, this ratio had changed 1 to 14.

Several factors appear to be involved in this swing. The importance of base courses to the service lives of roads is more generally appreciated than was the case a few years ago and planned stage construction is becoming more and more a common practice. Add to these that

frost action is one of the major causes of road damage in our climate, and that salt stabilized mats have in practice appeared to have definite resistance thereto, and the present trend is fairly well explained.

Ettinger: Stabilization fits well into the building of higher type surfaces. The stabilized roads can be held over for one or several years until such time as funds are available for the construction of higher types of surfaces. When such surfaces are placed on stabilized bases, they will be more durable, better roads than if they had been placed on unstabilized base material.

DESIGN

Rowat: The mixed aggregate used in the construction of stabilized bases in Quebec province, conforms to the following requirements:

Liquid limit .....under 25
Plasticity index 3-5
Centrifuge moisture equivalent.....under 20
Fraction passing the 200 mesh sieve not to exceed one half of the 40 mesh fraction.

Salt is used at the rate of 25 lb. per cu. yd. of aggregate. Both rock salt from Nova Scotia and evaporated fine salt from Ontario are used, the former in the east and the latter in the west of the province. Similar results have been obtained with both types of salt.

The application of bituminous tops to salt stabilized bases has in some cases been delayed until several months after the base is laid, and in other cases the asphalt surface has been placed with a seasoning period of only a few days. In the first case, the salt stabilized surface has served very satisfactorily as a wearing course during the interval.

As a general principle, we favor the longer seasoning period as affording an opportunity to correct local defects in the stabilization, but so far as present experience goes, the shorter seasoning period appears entirely satisfactory. It is, of course, too early to reach any definite verdict.

In the province of Nova Scotia an innovation in highway construction methods involving the use of salt, is directed towards the control of frost action and has now been practiced for two successive seasons.

The process consists in treating the sub-base course with rock salt at the approximate rate of 20 tons per mile of 20 ft. road. In special cases this may even be doubled. This treatment which undoubtedly increases the degree of compaction obtainable in the sub-base, and thus increases its bearing capacity, is based on the following considerations:

(1) That frost heaves usually originate in the sub-base course.
(2) That winter temperatures in the sub-base course, while below 32°F, rarely fall below 10-15°F.
(3) That it is the soil moisture, and not the soil itself which freezes.
(4) That the freezing point of the soil moisture in the sub-base can be reduced below the normal winter temperature range by treatment of this course with sufficient salt.

This plan of sub-base treatment has been applied on practically all new construction in the Nova Scotia provincial highway system during the past two years, and appears to be effective.

The experimental work of Mr. A. L. Pidous at McGill University in 1936 showed that 2.5 per cent salt will protect clay soils having a moisture content of 25 per cent, based on dry soil weight, against frost down to a temperature of 10°F, and the present work in Nova Scotia represents the practical field application of this finding.

Ettinger: In a County in Central New York State roads are built in a two-year stage. The grading, drainage and base courses are put down during the first summer. The base courses are built of either run of bank gravels or limestone aggregate bound with limestone dust in a

way similar to waterbound construction. Frequently the stone aggregate is bound with a mixture of limestone screenings and shoulder binder. In all cases, the materials are properly graded and the soil binders are of a quality to insure a satisfactory stable mixture.

Salt is placed in all of these bases generally in amounts of 2½ to 3 lb. per sq. yd. Care is taken to compact the bases when there is sufficient moisture to bind the stabilized materials tight. These bases will then serve as a wearing course for the remainder of the summer, and all of the following summer. During the third summer after completion, the bases will receive a bituminous top. From the time of completion of the salt base, until the bituminous surface is laid, two years hence, no salt is added and in most cases little if any machine maintenance is needed. The original, set up by the salt treatment, produces a hard, rugged surface which resists wear and weather. In many cases it was not even necessary to machine the base course surfaces prior to laying the bituminous pavement.

Downey: The Maintenance Division of the Michigan State Highway Department started experimenting this year (1939) with a type of shallow or surface stabilization using rock salt as an admixture to bind the loose material to the surface of gravel roads on the State system in Livingston County. On the work done to date the procedure has been extremely simple requiring a minimum of new material and equipment.

The loose gravel on the shoulders and in the ditches was collected and windrowed by means of a small shoulder maintainer. The quantity of material was estimated and sufficient soil binder was added to raise the quantity passing No. 200 sieve in the soil aggregate mixture to approximately 16 per cent. Rock salt was then added to the pile in quantities varying from two to four tons to the mile depending upon the quantity of gravel on hand. The materials were mixed dry until the clay was pulverized and thoroughly mixed through the gravel, this work being done with a shoulder maintainer to prevent material getting out on the traveled road.

The roadway was then wet down until it was soft enough to remove all holes and chatter bumps by scraping with an under truck blade and the material accumulated by this operation was pushed out to the shoulders to the dry mix. The entire pile was then brought in by means of under truck blades in successive layers until all the material was spread across the road. The blade bringing in the material was under the sprinkler truck so water was continuously added keeping the mix wet throughout the spreading. Another blade was used to shape the road so it would be smooth and have the proper crown.

When the material was all on the road it was left to traffic until it had hardened, usually a period of about six hours, after which the blade was put over the section to remove any loose bits of gravel and the whole thing was wet down again. This brought about a sealing of the surface which was hard, smooth and tight.

The road was used until it showed a tendency to ravel or get dusty, which time varied depending on the weather, from ten days to three weeks, when an application of about two tons of calcium chloride to the mile was made. On the heavier traveled sections successive applications of calcium chloride had to be made during the dry part of the summer so that on US-23 a total of six tons per mile was used, but on M-36 in the same county which was not completed until mid-July two tons of calcium has been used over the greater portion of the road.

These roads remained smooth and did not materially soften during or after rains, failing to show any tendency to rut or get soupy when wet. The throw of clay so often seen on gravel roads during rains was also missing. It is this ability to stay put and hold shape that makes this work

look attractive. Had the salt been placed deeper the benefits might have been more lasting; that is, the seal might have held for a longer period. In some cases so little gravel was on hand that the total layer of material was not more than one-half inch. This varied up to an inch and a half. In one case where a highly plastic clay was used it was found that the road was slippery after the first rain. Sand was added to this section while it was wet and the surface absorbed it and the condition did not occur again. In this way it was found that even if an error had been made in adding clay it might be corrected and the road retain its advantages of not softening.

The greatest difficulty encountered was trying to work the clay into a pulverized condition during the spring. As a remedy for this, clay is being dug from the banks and stock piled for a season of curing during the winter to facilitate pulverizing and mixing in the early spring.

The placing or laying of the material can be handled very rapidly. Two sprinkler trucks with blades and three inch pumps, and another truck with blade can place four miles per day. The cost of mixing the clay and digging the clay from the bank, of course, varies greatly depending on the length of haul and the season of the year.

It appears that by adding the salt early in the spring it will be possible to get it to a greater depth at little or no extra cost. At this time three or four tons to the mile would be added and the benefits would be expected to last over a longer period helping to keep the road hard enough to reduce damage from spring breaks.

The roads on which this method was tried are all in good condition at present, having remained hard and practically dustless through the fall season.

The construction division has gone ahead with their program of building bases for black top on several projects and rock salt and other chemical admixtures were used in the stabilized mix. This is all plant mix work and is closely supervised as to content of all materials and proper plasticity. However, on maintenance work in spite of the fact that it has not been possible to get such accurate control in the field, good roads have resulted.

Rowat: The central plant-mix method has been followed on all projects in Quebec. Potential sources of clay and gravel are sampled and tested by members of the central laboratory staff, and their recommendations determine the choice and the proportioning of materials. Control is exercised through appropriate field tests at the central plant.

Salt-stabilization practice in Manitoba has followed similar lines except that the salt is used in the form of brine hauled in tank trucks from the salt works at Neepawa, adjacent to the projects. In our opinion, road-mix methods are becoming obsolete.

MAINTENANCE

Ettinger: On an average, a salt road properly constructed and of good materials, requires one or two machine maintenances a year, generally in early spring, and possibly again in the fall. They also require on the average one application of one pound of salt per square yard, every spring at the time of the spring maintenance. Salt added each spring keeps these roads in excellent condition.

COSTS

Rowat: The cost of stabilization is considered to be the sum of expenditures for clay, salt, spreading and shaping, and compacting but not for the purchase of gravel.

From figures made available through the courtesy of the Quebec Department of Roads covering two separate projects, I have calculated the average stabilization costs per cubic yard of aggregate placed as 54 cents.

ECONOMICS OF STABILIZATION WITH CALCIUM CHLORIDE

By J. E. Moreland, *Tennessee Valley Authority*
Paul E. Glasgow, *County Engineer, Wayne County, Ohio*
F. L. Brown, *Superintendent of Highways, Washington County, New York*
George Farnsworth, *Superintendent of Highways, LaSalle County, Illinois*
A. L. Brown, *Assistant District Engineer, Michigan State Highway Department*
Rex M. Whitton, *Engineer of Maintenance, Missouri State Highway Department*
W. F. Abercrombie, *Engineer of Materials and Tests, Georgia State Highway Department*

Arranged by H. F. Clemmer, *Engineer of Materials, District of Columbia Highway Department*

SYNOPSIS

In this symposium George L. Farnsworth, related how LaSalle County, Illinois saved on maintenance of gravel roads by constructing 3 inch compacted stabilized gravel surfaces 20 feet wide at a cost of $2321.00 per mile. Prior to treatment, annual costs of maintenance had averaged $106.00 per mile for blading and $300.00 for gravel loss. Only half as much blading is now required. Three applications of calcium chloride per year cost about $265.00.

A. L. Brown notes several types of calcium chloride stabilization in use in Michigan. Base stabilization with 10 to 15 pounds of calcium chloride per ton of aggregate frequently precedes a bituminous surface treatment. Plant mixing of the material is usually preferred. Widening of the narrower paved roads is often accomplished by a 2-foot strip of stabilized gravel, 6 inches deep on each side of the road. The treatment costs about $650.00 per mile and is usually topped with a bituminous surface treatment.

The Missouri maintenance department has been experimenting with calcium chloride stabilization of clay gravel surfaces as a means of eliminating dust nuisance and improving the riding qualities of the surface. Rex M. Whitton described three recent projects, concluding that these objectives have been achieved and that the results justify continuation of the experiment to determine more definite conclusions both from a physical and economic standpoint.

W. F. Abercrombie of the Georgia Highway Department described road-mixing methods of stabilization using about 1.5 pounds of calcium chloride per square yard as an admixture and 0.5 pound later as surface treatment. Uncompacted soil was placed between 9-inch forms at a cost of from 8.4 to 9.6 cents per square yard. Calcium chloride cost 2.8 to 3.8 cents per square yard. The stabilized section was 19 feet wide with a compacted depth of 6 inches.

Francis L. Brown reported on the efforts of Washington County, New York, to take care of a large unpaved mileage by means of calcium chloride. Savings were realized in maintenance costs, $153.00 per mile for 1939 being the lowest yearly figure in the records.

Paul E. Glasgow described methods used in Wayne County, Ohio, in constructing a 21-mile section. Work included cleaning ditches, furnishing graded aggregate, binder soil and calcium chloride (1.5 pounds per square yard admixture and 0.5 pound surface treatment) and mixing. The complete course was 18 feet wide and 3 inches deep and total cost was about $2500.00 per mile. Mixing was accomplished with a traveling pugmill type machine. Covering the base with a bituminous treatment is the contemplated second step of this program.

According to James E. Moreland the road building activities of the T. V. A. are first, to provide access roads to construction sites; and second, to replace roads flooded by backwater from dams. A calcium chloride stabilized base will permit the placing of a bituminous surface treatment immediately after placing, which is of great importance if construction schedules for the dams are to be kept. Plant mix methods are usually preferred, except on small jobs. Costs of base stabilization varies from $1.38 per square yard for a 9-inch section, to $0.47 for a 6-inch section.

There are approximately 2,300,000 miles of unimproved secondary or "low-cost" roads in the United States, representing 80 per cent of the total road mileage. Considering traffic densities, however, expenditure for high type surfaces on the largest part of these roads is not practical. But, since these roads are vitally important to local educational, social, and business needs, there has been an increasing demand for the development of a low cost road construction program that would include their improvement. The applicability of calcium chloride treatment for this purpose is indicated in the following statements by the contributors to this symposium.

Glasgow: Approximately 45 per cent of the 50,000 population of Wayne County, Ohio is spread over the rural agricultural areas. Of the 1200 miles of roads in the County, 500 are on the county system, 500 on the township system, and 200 on the state system. The 500 miles on the county system consist of 440 miles of traffic-bound gravel, stone or slag roads, and but 60 miles of hard surface roads. We have found no satisfactory way of keeping the ordinary traffic bound roads in decent shape, regardless of cost, when the traffic gets up to 300 or more vehicles per day. Approximately 150 miles of our traffic bound roads are carrying from 150 to 500 or more vehicles per day. Our most pressing problem is, therefore, limited to this mileage. Following preliminary experiments during 1937, a contract for constructing 21 miles of calcium chloride stabilized base was let during 1939.

Whitton: In Missouri there has been for a number of years a growing demand for the elimination of dusty roads together with an increasing number of complaints about rough gravel roads through the dry summer months; especially is this true on those routes which carry a heavy traffic for this type of surface. At the present time there are 8,646 miles of gravel or crushed stone surfaced highways in Missouri under State maintenance. It is, of course, economically inadvisable and financially impossible to provide a dustless type surface for this entire mileage of gravel or crushed stone roads, but it is highly desirable to eliminate the dust on a part of it. It is, therefore, the general feeling in Missouri that there is a definite need for a low cost dustless surface.

Farnsworth: LaSalle County, located in the heart of Illinois's richest industrial and agricultural area is faced with the problem of developing a highway system which will enable both farmer and industrialist to market his products. In the past our policy was to link together the most important centers with hard-surfaced year-around roads, but once this was accomplished, the problem was what to do with the rapidly increasing mileage of secondary roads. Although the majority of these were gravel surfaced, the increased use of high-speed automobiles made the loose surface impractical and the next step was to decide upon an improved dustless low-cost type of surfacing which would include the major portion of our road system, and yet could be maintained within our present budget. After careful study of the various types available, we decided that the calcium chloride soil stabilized gravel road best fitted our conditions.

A. L. Brown: Michigan's climatic conditions are extremely varied, with temperature extremes ranging from −50° to 0° F. during the winter and 100° to 32° F. during the summer; and rainfall records show average monthly precipitations varying from 0.87 to over 4 in. Within this area are approximately 3000 miles of gravel trunkline roads which have traffic densities ranging from

over 1400 to less than 200 vehicles per day. Interspersed with the commercial truck traffic, local traffic and the usual inter-county traffic is a large tourist traffic which, according to the Public Roads Administration, expends over $300,000,000 annually in this State. This makes the tourist industry the second largest in the State, exceeded only by automobile manufacturing and means that special consideration must be given to the highway system in order to insure the continuation of this industry.

F. L. Brown: In Washington County, New York, up to 1937 a work relief program had decreased the ratio of paved to unpaved highways until it was evident that unless a reverse procedure was instituted, a serious drain on normal maintenance budgets might cause impairment to the service. Undertaking a year of maintenance in 1938, we set forth to attain two objectives: (1) to place all mileage in such a condition that it could be easily maintained until financial conditions would allow the return of a normal ratio of paved to unpaved mileage; and (2) to adapt our program to the public demand for smooth, dustless roads at a minimum of expense. To accomplish the objective with regard to gravel surfaces, a program of calcium chloride stabilization was adopted.

Moreland: The program of the Tennessee Valley Authority involves the construction of access roads to various dams under construction and the relocation or reconstruction of roads which are to be flooded by back-water from the dams. The policy of replacing with equivalent facilities dictates the type of surface on replacement roads, and the transportation demands determine the type of access roads. In both classes there are numerous projects on which it is permissible to use bases constructed of local materials such as crushed stone and gravel. This introduced the problem of utilizing these materials in an economical manner and producing a base of adequate strength which can receive a bituminous surface immediately after placing, since the construction schedules of work do not permit stage construction with long intervals during which traffic may consolidate the base and develop any weakness which may occur. Naturally a mixture of materials having a high density is called for, and our studies indicated that the methods of designing which are generally associated with the use of calcium chloride would be the most suitable.

Calcium chloride soil stabilization has been applied to base stabilization, partial surface stabilization, and complete surface stabilization. Accordingly it is necessary to discuss the economics of the various types of calcium chloride stabilization separately. Broadly, however, the general topic may be divided into two fields—base and surface construction; in the first case the ultimate surface will probably be a bituminous mat or concrete pavement, while in the latter case the calcium chloride stabilized surface constitutes the roadway wearing course. Several of the contributors to this symposium have offered discussions relating to both phases, while other contributors, due to prevailing local conditions, have confined their discussions to either one or to the other. The economic factors of the two construction types, as effected by calcium chloride are discussed as follows:

ECONOMY OF CALCIUM CHLORIDE IN STABILIZED BASE CONSTRUCTION

Subgrade

Moreland: In all of the T. V. A. projects precautions were taken to secure a sound, well drained subgrade. Wet spots were corrected by ditch or by tile under-

drains. Unsuitable materials were either replaced or treated to improve the quality. It is of course essential that the subgrade be firm and unyielding for any type of base to be successful. The stabilized base, when properly constructed, will retain its form, and will transfer superimposed loads to the subgrade, but it certainly cannot be expected to function satisfactorily if the subgrade is unstable.

Compaction

Moreland: In addition to the advantages of uniform and prearranged gradation of stabilized mixtures over water-bound or traffic-bound types of construction, the value of the incorporation of calcium chloride is also to be considered. An incorrectly proportioned mixture cannot be turned into a satisfactory one by the addition of calcium chloride, but it is undoubtedly true that the addition of calcium chloride to a well designed mixture will add to the density, stability and strength, and this additional value is obtained at a very low cost per square yard. Calcium chloride has been added to well maintained traffic-bound roads with good results, since the continuous moist condition caused by the presence of calcium chloride permits traffic to compact the surface effectively. The benefits derived from calcium chloride are more noticeable when it is incorporated in a well designed mixture during the process of construction. A base of satisfactory gradation to which calcium chloride has been added during construction, when properly compacted has a hardness and density which can only be explained by the fact that the presence of calcium chloride in the mixing water promotes compaction and increases the density. This is because the addition of this material increases the surface tension of the liquid, and permits soil particles to be moistened by thinner films of water. In *Public Roads* May, 1936, Messrs. Hogentogler and Willis make this statement: "Treatment with calcium chloride . . . . effects a decrease in the volume change and an increase in the density and stability of graded road mixtures." A progress report of the Project Committee on Stabilized Soil Road surfaces, issued by the Highway Research Board in 1935, contains the following statement: "The moisture film cohesion furnished by calcium chloride is more stable than that furnished by plain water, due to the combined effect of lower vapor pressure and hydroscopic property."

Clemmer: The research reported by the Public Roads Administration in the November, 1939 issue of *Public Roads* refers to outdoor circular track experiments in which various aggregates and admixtures all were used in the construction of base courses; which, following compaction, were covered with a light bituminous surface. The completed sections, as in the case of the bases, were subjected to distributed traffic wheel loadings, with the general surface conditions being noted periodically. The experiments were conducted in such a manner that it was practicable to control the water elevation and thus study the performance of the base under varying degrees of saturation.

With reference to compaction prior to application of the bituminous surfacing the following is quoted: "Compaction was continued on the top layer (of a two course base)until no further subsidence was noted and all sections were in suitable condition for testing. . . . . The effect of the chemical admixtures on the compaction of the graded materials is shown by the behavior of the test sections during the initial compaction period. Track 1, which contained calcium chloride, reached a condition considered suitable for starting the test at

somewhat less than one-third the wheel trips required to produce a similar condition in track 3 of the plain mixture." The numbers of trips were 18,200 for the calcium chloride track and 60,000 for the plain track.

A. L. Brown: The admixture of 10 to 15 lb. of calcium chloride per ton of gravel conserves the moisture and makes possible a greater density under compaction.

Surface Stability of Bituminous Mats

Clemmer: Quoting from the November 1939 issue of *Public Roads* . . . . "After this phase (testing of the respective bases without a bituminous surface) of testing had been completed, the sections were reshaped and trimmed smooth. A prime consisting of 0.3 gal. per sq. yd. of light tar was applied and allowed to cure. A surface treatment consisting of 0.4 gal. of hot bituminous material and a covering of 50 lb. per sq. yd. of stone of ¾ in. maximum size was constructed. The treatment was consolidated by additional distributed traffic until the surface was well sealed and showed no movement." "New initial or zero displacement readings were taken after the application and compaction of the bituminous surface and the record from that time on or from 171,200 wheel-trips to the end of the test indicates the behavior of the calcium chloride treated materials when acting solely as base courses."

"The materials in all sections of track 1 (calcium chloride) gave good service and showed little movement as base courses even under the very severe test conditions imposed by maintaining the water elevation at 2½ in. At 261,200 wheel-trips, or 90,000 wheel-trips after the start of concentrated traffic and 60,000 wheel trips after the water had been raised to the 2½ in. level, the average vertical displacement of the surface on all the calcium chloride sections was less than 0.05 in. and the maximum amount of rutting was 0.09 in."

Glasgow: The first step in our plan is to convert the traffic bound surfaces on our heavier travelled roads, into low-cost stable bases; the second step is to apply a 1½ to 2-in. bituminous treatment of the roadmix type. During 1937, we placed with our own forces a calcium chloride stabilized base on a two mile section of a county system road having a daily traffic of approximately 500 vehicles. The plasticity index for the final mix on this section was above 10. This base came through the winter of 1937-38 in excellent shape and was bituminous surfaced in 1938. The finished section, while developing a few failures (possibly on account of the high plasticity index of the base) during the winter of 1938-39 encouraged us to let a unit price contract for the construction of the 21 miles of various sections of county system roads.



Figure 1. Uniform spreading of Calcium Chloride, Wayne County, Ohio

Moreland: *Hiwassee Access Road.* This road extends from Hiwassee dam site to the nearest railroad twelve miles away. It was designed for the transportation of all equipment both permanent and temporary, all materials except the concrete aggregates, and a large percentage of the labor. The cement was transported over this road in vehicles holding 60 bbl., or approximately 12 tons of cement. The heaviest individual load was estimated to be 60 tons. The aver-

age traffic during construction has been approximately 1,000 vehicles per day. The base was built in two layers to a total thickness of 9 in. and immediately after construction was given a bituminous prime and seal coat of hot asphalt, with 50 lb. of stone chips rolled into the asphalt. The base material which was composed of crushed stone, local soil, water and calcium chloride, was mixed in a concrete mixer. There have been no failures of the base except a minor one at a point where the subgrade was not well drained.

Alabama State Highway No. 1. A section of this highway, four miles in length, was built to replace an existing road which has since been flooded. The road replaced was composed of a crushed stone and gravel base, with a bituminous prime and mixed in place surface. The traffic on this section was 1,000 or more vehicles per day, including heavy truck traffic. The road which we constructed to replace this was composed of a stabilized base 6 in. thick, with a bituminous prime and 1 in. mixed in place bituminous surface. The base was of crushed stone, local soil, calcium chloride and water, mixed on the subgrade in two layers with a heavy grader. No failures were encountered during the past winter, which was the first winter after construction.

Chicamaugua Access Road. This road extends from Chicamaugua dam site to Tennessee Highway 58, one mile away, and is used almost entirely for passenger traffic to the dam consisting of approximately 1200 vehicles per day. On this road we constructed a stabilized base 5 in. thick, and applied the bituminous prime and hot bituminous seal with 50 lb. of stone chips per square yard. This base is composed of crushed stone, local soil, calcium chloride and water, mixed in place in two layers. The road has been in use for three years with no base failures, and only moderate maintenance on the surface.

Kentucky Access Road. This road extends from the site of the Kentucky dam near Gilbertsville, Kentucky, to U. S. Highway 68, seven miles away. It is being constructed to provide transportation for labor and visitors to the dam during construction. The traffic will probably be 1200 vehicles per day. The base course is 6 in. thick, with a bituminous prime and hot bituminous seal coat. The materials of the base are bank gravel, sand, calcium chloride and water. The base was mixed in place with a heavy grader in two layers. It has only recently been finished so there is no record of the maintenance.

Watts Bar Access Road. This road extends from the site of Watts Bar dam to the nearest serviceable road two miles away. It will provide transportation for part of the labor used on the dam, and for supplies to the construction camp. The traffic will be some 800 vehicles per day, including a fair proportion of trucks. The base is of stabilized material 6 in. thick, with a bituminous prime and hot bituminous seal coat added. The materials in this case are gravel, calcium chloride and water. The base was recently completed, and has had no maintenance cost.

A. L. Brown: The Michigan plan for base course construction anticipates the application of a higher type bituminous surface. The final road as designed and as proven by experience, has provided the economical, low maintenance cost type of surface that must be used to keep up with the rapid increase in traffic on the Michigan State highways. If the oil aggregate wearing course is deferred, the gravel road can be treated with a surface application of calcium chloride during the dry periods, which will require a minimum of maintenance for one, two or three years. In some cases it has proved practical to apply a bituminous surface treatment to protect the base for one or

two years, and in this case a prime coat is not used when the oil aggregate is applied.

Summary on Base Stabilization

Moreland: The following benefits are received from the inclusion of calcium chloride in the soil mixture:

1. Greater ease of manipulation, since the mixing water does not evaporate to an inconvenient degree.
2. Greater density and cohesion, resulting in added strength per inch.
3. Insurance against premature drying after completion, with added maintenance costs.
4. An apparent aid to the absorption of bituminous priming materials.

In conclusion, experience has shown that the intelligent use of the accepted criteria and methods for using local materials in a stabilized base will result in real economies on many road projects, and that the addition of the proper quantity of calcium chloride is amply justified by the increased efficiency of the materials used and reduction in maintenance costs during construction.

A. L. Brown: The Michigan State Highway Department, faced with the problem of improving traffic conditions, and of satisfying public complaints; confronted with problems caused by a wide range in maintenance season; faced by a wide change in climatic conditions; forced because of financial conditions to use the available local materials; finding that the increased traffic in industrial areas has made obsolete the recently built hard surfaced roads; can point with pride to the economy and improved conditions resulting from soil-aggregate stabilization with calcium chloride.

ECONOMY OF CALCIUM CHLORIDE IN STABILIZED SURFACE CONSTRUCTION

Many conditions exist where a stabilized surface is by far the most economical type of construction. It should likewise be pointed out that the major portion of all "base" projects serve as surfaces for several years following their construction. Although many of the economic benefits pertaining to bases likewise apply to surfaces, the following section is devoted to economics in the construction of stabilized surfaces as effected by the use of calcium chloride.

A. L. Brown: Any procedure that tends toward saving of material and decreasing of maintenance costs should warrant our earnest attention. Every $10.00 saved in maintenance on each mile of the 3000 miles of gravel road in Michigan's trunk-line system, a total of $30,000, means enough to place 20 miles of bituminous surface treatment on prepared base stabilization.

Farnsworth: LaSalle County, Illinois has 400 miles of gravel roads. To maintain these roads in the past, twelve motor patrol graders were kept at work at an annual cost of $106 per mile. Added to this was the cost of replacing gravel lost as dust, forced into the subgrade and thrown from the road by traffic. The annual loss of material from our principal gravel roads amounted to 200 cu. yd. per mile, based on our total replacement of material every five years. This gravel cost the county $1.50 per cu. yd. or $300 per mile delivered on the road. The total annual maintenance cost for roads which were dusty during the summer season and unstable in the early spring was, therefore, $406 per mile.

Under our present system of maintenance, blading costs have been considerably reduced. Where the principal gravel

roads had to be bladed at least once or twice each week, these roads, stabilized and maintained with calcium chloride, now require blading on an average of only once each month. This means that each patrolman can now maintain a greater mileage of roads and otherwise devote former blading time to other maintenance work.

The replacement of gravel formerly lost has not been measurable and we well feel that this is no longer an important factor with the dust kept in the road as a binder by the moisture supplied through calcium chloride and with a tight surface resulting with no loose gravel.

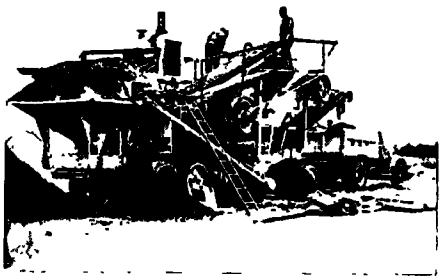


Figure 2. A plant arrangement for preparing stabilized mixtures. LaSalle County, Illinois

With the use of stable gravel maintained with calcium chloride we have almost eliminated the gravel loss, and estimate a 50 per cent saving in blading costs.

The cost of our annual calcium chloride treatment, at 10 tons per mile, is $265 which is actually paid for by the savings in gravel replacements and decreased blade maintenance. We also are realizing other advantages gained by this type of surface which are not directly measurable in dollars and cents, such as increased safety, health and public satisfaction.

Whitton: In the summer and fall of 1938, the success of nearby state highway departments with the use of calcium chloride on surface-stabilized clay-aggregate roads was noted, and it was then decided to do some experimenting along the same lines in Missouri. Three widely separated sections surfaced with loose aggregate, and representative of the different conditions encountered in Missouri were selected for the clay-aggregate surface stabilization experiments using calcium chloride on the surface.

The first section is located on State Route 4 in Gentry County in northwest Missouri, and is 6.1 miles in length. This section was selected because it was the only gravel surfaced section on this route for a considerable distance in either direction; and since the average daily traffic count was approximately 250 vehicles, the surface became very dusty and rough during the summer months. The reconstructed stabilized section was composed of existing loose roadway gravel, together with binder soil from the roadway ditches, moisture and calcium chloride, compacted by traffic to a thickness of approximately 4 in.

The second experimental section 4.6 miles in length, is located on State Route 20 in Saline County in west central Missouri, and is a portion of a 29-mile section of crushed stone surface. This section was selected because of the exorbitant crushed stone replacement cost and, also because of the extremely dusty condition of the road at certain periods of the year. The crushed stone replacement cost on this particular section of road for the past four years has been $510 per mile per year, while the average replacement cost for this territory, as a whole, has been $135 per mile per year. The average daily traffic is approximately 200 vehicles. The reconstructed stabilized section consists of crushed limestone mixed with subgrade binder soil of the Marshall soil series, together with moisture and calcium chloride. Following thorough mixing with a Seaman Soil

Tiller and spreading with motor graders, the completed roadway was compacted by traffic.

The third section is located on State Route 53 in Dunklin County in the extreme southeast portion of Missouri, and was selected because of the extremely rough surface conditions which prevailed through the summer months. This condition was caused primarily by the sandy condition of the soil and the lack of proper binder and moisture. During the spring when sand storms occur, sand drifts in from the fields, filling up the ditches and adding additional fines to the surfacing material on the roadway. During the past few years both quantity and speed of traffic have increased very materially, thus resulting in additional difficulties in maintaining a smooth riding surface. The surface of the roadway would become very corrugated, full of pot holes and extremely dusty during dry periods of the year. The average daily traffic on this section is now approximately 150 vehicles. The reconstructed stabilized section consists of added binder soil, roadway gravel free of oversize material, calcium chloride and moisture; which, following thorough mixing, was compacted by traffic.

Considering the three projects as a whole, it is felt that the first year's results have been satisfactory. The dust nuisance has been eliminated entirely on the latter two sections and to a great extent on the first section, and it has been possible to maintain a much smoother surface than had been possible in the past on these sections through the dry periods. Comments from the public have been highly complimentary. Results also seem to justify the continuation of the experiments of these particular sections in order to arrive at more definite conclusions, from an economical as well as a physical viewpoint, based on observations made over a longer period of time.

Abercrombie: The use of calcium chloride stabilization is comparatively new in Georgia. During the late summer of 1937 a two mile experimental section was built, after which, during 1938, three Federal Aid projects were constructed. Two of these latter projects consisted of pebble soil and the third of sand-clay. The typical sections were 19 ft. wide by 6 in. compacted depth. We have been studying two of these projects with respect to soil loss and to date have made two readings. The first reading was made within two months after the projects were finished and the second reading about six months later. These readings showed practically no loss; however, it is felt the time interval is too short to give conclusive results.

F. L. Brown: In Washington County, New York, the normal program of gravel road maintenance in the past has been to patrol the gravel roads with power driven grader patrols using two units. A simple analysis showed quickly the cost involved.

2 motor patrols at $2.70 per hr., each ..... $43.20 per 8-hr. day
For a normal 5-day week, 7 months season..... $6480.00 per season
Cost per mile per year (on approx. 95 mile basis).... $68.20

This service guaranteed that the roads would be leveled off about once a week, however, it took only a day or two until "washboarding" and other evidence of roughness appeared. In addition, dusty roads were ever present.

The cost of gravel blown off the road is another item of expense which may run as high as 300 cu. yd. per mile per year.

In considering Washington County's program of stabilization it should be noted that roads having gravel surfaces are classified roughly as follows:

1. Well graded surfaces to be paved shortly.

2. Well graded surfaces suitable if stabilized.
3. Poorly graded surfaces to be reconstructed.
4. Poorly graded surfaces, reconstruction not justified.

Groups 1 and 2 deserved the most immediate attention. The use of large quantities of calcium chloride in Group 1 was not thought to be justifiable but Group 2 deserved all possible care. To Groups 1 and 2 enough material was added to allow grading to a parabolic crown from edge of shoulder to edge of shoulder. From 3 to 8 tons per mile of calcium chloride was added. Roads in Group 2 were treated with at least 8 tons per mile, after which rolling was initiated and continued until consolidation was effected. No water was added; natural rains were depended upon to maintain the moisture ratio at the optimum amount at the time of application of calcium chloride. To Group 3 calcium chloride was applied only as a dust layer. An attempt was made with Group 4 to remove high shoulders and establish adequate drainage before gravel was added to construct a suitable crown. Immediately thereafter the roads were treated similarly to Groups 1 and 2.

The economy of this program is illustrated in Table 1, which may be best emphasized when it is noted that in 1939, the second year of the program, blading and graveling costs were cut in half. In addition to this, a dustless, smooth road serves the farmer to his better satisfaction, while he awaits the day of the hard surfacing of his road.

Summary on Surface Stabilization

A. L. Brown: We have saved the loss of wearing material from the road surface and eliminated a large part of costly hand patching. We have greatly reduced the need for costly blading. And we have lowered the traffic count on the main highways by diverting traffic to the secondary roads where the motorist can now travel in safety and comfort.

Indirect economic results as obtained for our motorists are:

1. Increase of safety and prevention of accidents by eliminating dust and loose material hazards.
2. Financial and health saving to property owners adjacent to the road.
3. Saving on the upkeep of automobiles through improved stabilized roads.
4. Early lifting of weight restrictions.

Farnsworth: The program of LaSalle County has proved satisfactory and we

TABLE 1 (F. L. BROWN)
COST OF GRAVEL ROAD MAINTENANCE

	1934	1935*	1936*	1937	1938	1939
Blading cost per mile per year	81.50	40.50	48.50	79 00	54.50	38.20
Gravel replacement per mile per year	136.00	55.50	84.80	168.00	87.50	36.10
Calcium chloride treatment per mile per year	.....	.....	.....	.....	128.00	79 50
Total	217.50	96.00	133.30	247.00	270.00	153.80

* The slump occurring in 1935 and 1936 was the result of an economy move by the Board of Supervisors who reduced greatly all normal appropriations and devoted much of the year's program to work relief. The normal program was resumed in 1937.

have increased the original 21.5 miles constructed in 1937 to approximately 70 miles of stabilized calcium chloride roads and we plan to add to this mileage each



Figure 3. An inexpensive method of securing uniform distribution, of stabilized mixture. LaSalle County, Illinois.

year. Thus we feel that we are accomplishing our original purpose; to develop a highway system that is economically sound, spreading improved highway benefits over a large area and yet maintaining them with our present maintenance budget.

F. L. Brown: The conclusions to be drawn from our program may be summarized as:

1. Calcium chloride stabilization will give to properly graded gravel roads a smooth, hard, dustless surface.
2. Calcium chloride stabilized roads will reduce normal maintenance costs for this class of road.
3. There is a distinct reduction in blading and gravel replacement costs after stabilization. Before stabilization the cost of blading ran about $68.20 per mile per year. This was reduced so that graders were in use on maintenance for less than half the working season, freeing these machines for construction operations.

MINUTES OF BUSINESS MEETING

HIGHWAY RESEARCH BOARD

December 6, 1939

The meeting was called to order with Chairman Marsh presiding.

ATTENDANCE

Executive Committee

Burton W. Marsh, Chairman
A. T. Goldbeck
C. M. Upham
F. C. Lang
T. R. Agg
H. C. Dickinson
W. A. Van Duzer
R. W. Crum, Director

Member Organizations

American Automobile Association
Burton W. Marsh

American Concrete Institute
P. H. Bates, H. J. Gilkey

American Road Builders' Association
Charles M. Upham

American Society for Testing Materials
F. E. Richart

The Asphalt Institute
J. E. Pennybacker

Association of Asphalt Paving Technologists
W. J. Emmons

National Bureau of Standards
P. H. Bates

Institute of Traffic Engineers
L. W. McIntyre, R. L. Morrison

National Crushed Stone Association
A. T. Goldbeck

Portland Cement Association
M. D. Catton

Society of American Military Engineers
H. C. Whitehurst

Western Society of Engineers
A. N. Talbot

Forest Service
T. W. Norcross, H. R. Jones

Minutes of Meeting of November 30, 1938.

Motion: That the Minutes of the meeting of November 30, 1938, be approved as printed in the Proceedings of the Eighteenth Annual Meeting.

Adopted

Amendments to By-Laws.

The Director reported the result of the letter ballot on the Amendments to the By-Laws as follows:

On Dues.....Affirmative 24; Negative, 1; Not Voting, 8.
On Associates..Affirmative 25; Negative, 0; Not Voting, 8.

Membership.

The Director reported the result of the letter ballot on admission to membership of the Forest Service, U. S. Department of Agriculture, as follows:

Affirmative	24
Signed but not marked "for" or "against"	2
Negative	0

It was also reported that of the 33 organizations listed as members prior to the change in the By-Laws, 24 are in good standing at this date, one is no longer in existence, two still have the matter under consideration, and six have dropped out. Some of the last group are expected to return later.

Report of Nominating Committee.

The Nominating Committee, consisting of W. J. Emmons, Chairman, P. H. Bates, T. H. MacDonald, H. G. Shirley and H. C. Whitehurst reported the nominations of Burton W. Marsh, Charles M. Upham and Lion Gardiner for membership on the Executive Committee.

Motion: That the report of the Nominating Committee be approved and the

Director be instructed to cast a unanimous ballot for the nominees.

Adopted

Special Projects.

Light as Affecting Highway Travel at Night: Chairman Marsh reported that substantial progress is being made on the research program recommended by the Working Committee. Several phases of the program are being carried out by the Public Roads Administration and a progress report is to be presented at this Annual Meeting.

Lecture Course in Highway Economics: The Director reported that a lecture course in highway economics sponsored jointly by the Iowa State College and the Highway Research Board with the cooperation of the Public Roads Administration is to be given at Ames, Iowa, January 8 to 12, 1940. The purpose is to make available to highway administrators and engineers engaged in the planning of future improvement and maintenance programs a course of lectures, of graduate school grade, on the fundamentals of economics which apply in the operation of highway systems. This is expected to be the first of a series of similar lecture courses to be held at regional centers throughout the United States.

Highway Research Census. The Director reported that, in cooperation with the American Association of State Highway Officials a census of research projects and reports had been published in mimeograph form for preliminary distribution among the contributors of the information. The data are now being brought up to date and corrected for early publication.

Memorials.

The Director was instructed to prepare for inclusion in the Proceedings suitable memorials on the passing of our friends and co-workers, George F. Schlesinger and Albin L. Gemeny.

Appreciation.

The Director was instructed to send letters of appreciation in the name of the Board to the participants in the program of the Nineteenth Annual Meeting and to the officials of the National Academy of Sciences and the National Research Council.

No further business appearing the meeting was duly adjourned.

R. W. CRUM,
Director.

BY-LAWS OF THE HIGHWAY RESEARCH BOARD

Effective July 1, 1939

1. *Name:*
   Highway Research Board.
2. *Purposes:*
   (a) To assist in outlining a comprehensive national program of highway research and coordinating activities thereunder.
   (b) To organize committees for specific problems.
   (c) To deal with ways and means.
   (d) To act in a general advisory capacity.
3. *Members and Associates:*
   (a) Members shall be organizations or groups engaged in research related to the highway industry, associations, societies, institutes, or foundations whose activities are educational in character or associations or institutes that are interested in the technical development within an industry or group of industries, each acting through one duly appointed representative, with authority to name an alternate, serving until appointment of his successor. New organization members shall be elected by letter ballot of the Board upon recommendation by the Executive Committee. Qualification shall be completed within three months by appointment of a representative and payment of dues which shall be pro rated for the nearest quarter period of the fiscal year of the Board. Members shall be classified and pay annual dues as follows:
      1. Agencies of the United States Federal Govenment—no dues.
      2. States, county or municipal governments, colleges, universities or other educational institutions—$10.00.
      3. Associations, societies, institutes or foundations whose activities are educational in character—$0.50 per $1,000 of normal annual income, but not less than $5.00 nor more than $50.00.
      4. Associations or institutes that are interested in the technical development within an industry or group of industries—$2.00 for each $1,000 of normal expenditure for purposes related to highways during the preceding year, except that the dues shall not be less than $100 nor more than $1,500.
   (b) Associates shall be firms, corporations and individuals qualified by interest and activity in highway research, who desire to further the work of the Board through annual contributions to its support. The annual fees shall be:

Firms and corporations	$50.00
Individuals	$ 3.00

Associates shall be entitled to all of the privileges of the Board except the right to vote. Firms or corporations accepted as associates shall designate a representative and an alternate from their research or technical organization. Associates shall be elected by the Executive Committee.

4. *Officers:*
   Chairman, Vice-Chairman and Director, who shall perform the usual functions. The Chairman and Vice-Chairman shall be elected annually by the Executive Committee from among the members of the Executive Committee, to serve one year from the annual meeting or until their suucessors qualify.
5. *Director:*
   Shall be engaged by Executive Committee and serve at the will of that committee; shall be paid a salary fixed by that committee; shall be the executive officer of the Board; shall be a professional engineer; shall devote his whole time to the duties of his office; shall account to the Executive Committee for all monies received and disbursed.
6. *Administrative Committees:*
   (a) *Executive:* The Executive Committee shall consist of eleven members, two of whom shall be ex-officio—an officer of the Division of Engineering and Industrial Research of the National Research Council and the Commissioner of the Public Roads Administration, Federal Works Agency—and nine shall be elected by the Board at the annual meeting. Three members shall be elected each year to serve for three years.

   Should a vacancy exist in the Executive Committee, it shall be filled by election by the Executive Committee.

   (b) *Ways and Means:* Three members appointed by the Chairman.

   Duties: Those indicated by the titles and such others as may be assigned by the Board. The Executive Committee shall also aid the Chairman and Director in carrying out instructions of the Board and in formulating projects to be presented to the Board; shall act for the Board in routine matters, and shall control publicity.
7. *Research Committees:*
   Shall be created by the Board, as deemed necessary, with the approval of the Division of Engineering. Members shall be appointed by the Chairman of the Board with approval of the Executive Committee, and need not be limited to members of the Board.

   The term of service of members of Research Committees shall be limited to one year, but they are eligible to reappointment.
8. *Meetings:*
   An annual meeting shall be held at the time and place designated by the Executive Committee. Other meetings at call of Executive Committee on twenty days' notice.
9. *Amendments:*
   May be made by a majority of the Board, on thirty days' notice, by letter ballot.

MEMORIALS

GEORGE FEURLE SCHLESINGER

Resolution of the Highway Research Board:

With profound sorrow the members of the Highway Research Board and their associates record the death on December 1, 1939 of their friend and fellow worker, George F. Schlesinger.

Mr. Schlesinger graduated from Ohio State University in 1907. His succeeding years of practice as a faculty member of Ohio State University, as a constructing engineer, as Chief Engineer and Director of the Ohio State Department of Public Works, and as Chief Engineer and Managing Director of the National Paving Brick Association caused him to be highly esteemed by the engineering profession and industrial leaders alike.

In him business judgment and technical knowledge were so well blended with executive ability and sympathetic understanding that his advice became invaluable and his friendships remained true. We gratefully, therefore, memorize the many years of his unselfish work and effective efforts to advance the interests of the Board and its committee activities.

Be it therefore resolved that this expression of appreciation be sent to the family of Mr. Schlesinger, and to the President of the National Paving Brick Association and that it be printed in the Annual Proceedings of the Highway Research Board.

ALBIN LEROY GEMENY

Resolution of the Highway Research Board:

The Highway Research Board at its Annual Meeting on December 6, 1939, wishes to express on behalf of its members and associated workers a deep sense of personal loss in the untimely death of Albin Leroy Gemeny, a friend and co-worker of many years.

After graduation from the University of Virginia in 1911 his career as railway construction engineer, valuation engineer with the Interstate Commerce Commission, bridge engineer Dominican Department of Public Works, engineer U. S. Bureau of Public Roads, engineer Mexican Highway Commission and finally senior structural engineer Public Roads Administration lead him to a position of acknowledged leadership in highway structural engineering. A profound student and brilliant engineer, his labors and wise counsel in the deliberations of the Department of Design of the Highway Research Board, the American Association of State Highway Officials and other agencies serving public interests remain of nation-wide importance and unusual value to his professional associates. His colleagues realize their great loss in being deprived of his welcome presence as a kind and understanding friend but are grateful for the privilege of their long association with him.

Be it therefore resolved that this expression of appreciation be sent to the family of Mr. Gemeny and to the Commissioner of Public Roads and that it be printed in the Annual Proceedings of the Highway Research Board.

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