DIVISION OF ENGINEERING AND INDUSTRIAL RESEARCH
NATIONAL RESEARCH COUNCIL

HIGHWAY RESEARCH BOARD

PROCEEDINGS
OF THE
SEVENTEENTH ANNUAL MEETING

Held at Washington, D. C.
November 30, December 1, 2, 3, 1937

EDITED BY
ROY W. CRUM
Director, Highway Research Board

WASHINGTON, D. C.

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 35 technical and commercial associations and organizations of national scope

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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Subcommittee No 3 "Measurement of Earth Pressure in the Underground"

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Methods of Compaction for Control and Construction and Their Effect Upon the Properties of Soils for Highway Purposes, R R Philippe, Chairman, E McD Moore, Kenneth B Woods

Practical Design of Drainage for Highways and Airports Exclusive of Bridges and Culverts, W J Schlick, Chairman, W B Worsham

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Stress Distribution in Earth Masses (a) Load due to own weight, (b) Load through pavements, (c) External Loads on Foundations, D P Krynine, Chairman, Robert G Hennes, William P Kimball, Nathan M Newmark, L A Palmer, Fred L Plummer, M G Spangler, Warner Tufts

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Subcommittee No 2 "Stress Distribution in Limited Earth Masses Stress Trajectories and Values of the Concentration Factor Close to the Boundary of a Limited Mass Pressures Against Retaining Walls Stability of Slopes" William P Kimball, D P Krynine, Nathan M Newmark

Subcommittee No 3 "Inter-reaction of Structures and Earth Masses 'Equivalent Masses' and 'Rigid Shells' Distribution of Stresses at the Base of a Structure Stresses in the Structure as Influenced by the Physical Properties of the Foundation" Fred L Plummer

Subcommittee No 4 "Influence of Plates, Rigid or Elastic, Between the Load and the Earth Mass Stresses in the Pavement as Influenced by the Physical Properties of the Foundation Deformation of the Intermediate Plate Flexible Pavements" D P Krynine, M G Spangler

Subcommittee No 5 "Stress Distribution Within Soil Samples Critical Analysis of the Consolidation Test and of the Shearing Test Principal Stresses Within a Sample" William P Kimball

Subcommittee No 6 "Introduction of Statistical Methods into the Stress Distribution Theories" Warner Tufts

Subcommittee No 7 "Stresses in Earth Masses Due to Vibration, Impact, and Suddenly Applied Loads"

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Subcommittee No 4 "Surface Reactions of Importance in Stabilization of Soils by Means of Portland Cement," Paul Rapp, Chairman, C A Hogentogler, Jr, F T Sheets (Alternate, M D Catton) Hans Winterkorn

Stabilized Roads, C A Hogentogler, Chairman, L L Allen, A R Brickler, F H Eno, J W Kushing, W H Mills, Jr, G A Rahn, Arthur R Smith, E A Willis, W B Worsham

Subcommittee No 1 "Design of Natural Soil Mixtures and Effect of Admixtures of Commercial Products Such as Crushed Stone, Gravel and Slag," E A Willis, Chairman, W P Brown, A T Goldbeck, Fred Hubbard, William Lavers, Stanton Walker

Subcommittee No 2 "Effect of Methods of Manipulation and Compaction, Treatment With Substances Involving Chemical Reactions Productive of Permanent Crystallization Such as Portland Cement, Calcium Silicate and Calcium Carbonate," Paul Rapp, Chairman

Subcommittee No 3 "Use of Bituminous Materials, Including Oils, Tars, Cut-Backs, and Emulsions, as Admixtures or for Special Treatment to Destroy the Colloidal Properties of Soils Productive of Detrimental Volume Change," L L Allen, Chairman

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REPORT OF THE DEPARTMENT OF HIGHWAY FINANCE

T H MacDonald *Chairman*

Chief U S Bureau of Public Roads

PROBLEMS OF FINANCIAL ADMINISTRATION WITH SPECIAL REFERENCE TO LOCAL ROADS AND STREETS *

SYNOPSIS

In 1936 approximately a quarter of a billion dollars, or one-fourth of total State motor vehicle collections, were distributed for local road and street purposes This was more than double the amount distributed in 1927 However the increase in the part of the total collections so distributed has only been 3 percent

The amount of State funds spent on State roads has also increased in this 10-year period, but by only 42 percent as compared with the 115 percent increase in local road apportionments Moreover, there has been an actual decrease of 17 percent in the share of total State taxes so used This discrepancy appears to result from a wholesale use of funds for other than highway purposes

The amounts and methods of distributing user taxes going to local units of government vary widely from State to State Distribution among the local units was found to be based on a variety of criteria, including population, area, vehicle registrations, valuation, tax collections, road mileage, and combinations of these factors In some States these funds are distributed equally among the local governments

It has been found that these methods of local road allocations often fail to reflect properly the needs of the highway system as a whole Economic allocation of funds requires that money be spent according to the needs of traffic, expressed in terms of the lowest possible total cost of transportation, which includes not only road costs but vehicle operating costs It is not merely total traffic which must be considered, but the concentration of this traffic In determining priority for expenditure it should be remembered that because of the integration of traffic on several road systems, it is advisable to improve the primary system first since it carries the largest amount of concentrated traffic and its improvement brings about increased travel and increased receipts for the support of local roads

It has been found that the spending of State funds by local governments is not always to best advantage because it is not properly controlled by the State Many local units do not comprise sufficient taxable wealth and highway activities to qualify them as logical highway administrative agencies

In the search for the proper scope for highway activities it is concluded that there may be a distinction between the highway administrative unit and the economic operating unit that the former may comprise several of the latter

Operating units which do not have sufficient taxable wealth and traffic may require consolidation before they are able to perform their functions economically Among other things there must be sufficient road work to allow efficient utilization of equipment, and sufficient appropriations to provide a competent engineering force

Rural county highway units may comprise large areas for economic operations, while the urban county, because of its wealth, population, and traffic, may properly be confined to a small area Because the urban county is usually part of a larger metropolitan area containing other counties, provision is needed for correlated action, in highway planning and in fixing priorities for improvement programs

Correction of the weaknesses of local highway administrative finance has been attempted by transfer of local roads to State control In the past 6 years 21 States have taken over 172,000 miles of local roads, constituting a 64 percent

* Based on a Report prepared for the Committee by Wilfred Owen, Research Assistant, Highway Research Board

increase in State mileage during that period Four States have eliminated all locally administered rural highways These have shifted the road burden from land to motor vehicles and from local government to the State

Although the road consolidation movement was precipitated by the recent economic depression as a means of relieving property of the road tax burden, it appears that the inherent failings of local governments have been underlying causes of the movement, for in 1936 ten States effected local road transfers to their State highway departments, a larger number than in any previous year

It is felt that the policy of Federal-aid appropriations for secondary roads, as well as the trend toward highway planning, will in many cases accentuate the movement for State administration of rural roads

Consideration of present and past methods of highway financial administration reveals slow progress toward a rational spending program Demands for increased highway appropriations have in many cases diverted attention from the need for wiser spending of what we have and more efficient managing of what we spend Design, construction and maintenance standards have kept reasonable pace with modern transport tempo, but policies of administration and finance remain essentially horse-drawn

Highway tax distribution and the administrative difficulties involved have been examined with particular reference to local application of State funds for highway purposes Last year more than a quarter of a billion dollars in State gasoline taxes and registration fees were set aside for roads and streets not on the State highway systems This money was 25 percent of the total motor vehicle tax collections for 1936 The large part of highway user taxes so distributed is an index of the need for studying methods of allocating such funds to local governments, for establishing an economic basis for shared taxes and State aid, and for inquiring into the uses to which these funds are now applied, the degree of financial control retained by the States, and the fiscal and managerial pitfalls into which both State and local governments spend their way

VEHICLE TAXES FOR LOCAL ROADS

That highway users should be charged in accordance with their utilization of highway facilities is the generally accepted theory upon which the gasoline tax and registration fee are established It appears to follow therefore that the distribution of such taxes to various parts of the highway system should reflect the relative traffic volumes which they carry

In the period of rapid highway expansion which paralleled the growth of motor vehicle travel, the theory that those who used the roads should pay for them was generally conceded, but financial pressure created by the need for a new system of main roads made it neither possible nor desirable to adopt the corollary that funds should be spent with exact regard to their origin With the progress of a primary system of highways which such concentrated finance made possible, however, there originated in both counties and municipalities a demand that some part of State tax collections be returned for local roads and streets Today the wide range in relative proportions of funds made available to local governments suggests no more scientific consideration than the loudness of these demands In 1936, 3 States returned more than half of their total highway user imposts to local units of government, 11 over one-third of such collections, and 5 States made no such allotments whatever Local roads in one State received 24 million dollars from State taxes, while in each of 10 other States less than a million dollars were distributed for highways in local jurisdictions

TAX DISTRIBUTION LAWS *

State laws governing the amount and basis of gasoline tax and registration fee distribution comprise a legal labyrinth which varies in complexity from State to State Two considerations are involved determination of the total which shall be distributed by the State, and the division of this sum among the various local units The total share going to local roads is generally expressed as a percentage of collections, a specific part of each tax levied, or a predetermined flat sum The allocation to each local unit may then be made according to the population, area, assessed valuation, road mileage, or on the basis of vehicle registrations or tax collections In the case of the registration fee, however, shares are often retained by each separate local unit at the time of collection, either as a fixed amount per registration or a percentage of total receipts

Although the total amount of motor vehicle taxes granted for local road purposes may have no relation to traffic needs originating on these systems, in a large number of States registration fees are allocated among the separate units with a regard for relative traffic potentialities Thus Arizona counties retain 50 cents for each original registration, while in Alabama 20 percent of total receipts from this source are used in the counties where the taxpayers reside In the case of the gasoline tax, however, not only does the original sum granted by the State have little bearing upon traffic volume and intensity, but also the allocations among individual local units are generally based upon formulas which are untenable Alabama, for example, distributes 3 cents of a 6-cent tax equally among its 67 counties, while New York counties receive 20 percent of collections according to the road mileage of each county In Tennessee one cent of the gas tax is distributed to the counties equally, ½ cent on county areas, and ½ cent according to county populations

When money is distributed equally among local road units which vary in size and stage of development, or on the basis of land areas and road mileage which bear no relation to traffic conditions, there is little chance that distribution will be economically justifiable Only by chance will highway income be in reasonable balance with the demand for funds Even population and assessed valuation may be poor indices of the proper share of taxes required by local governments for transport facilities Questionable practices of tax allocation accordingly help to make possible such variations in road expenditures as found in North Carolina before the State assumed control of all rural roads The annual road expenditure in one county was $14 per mile, while in another it was $688 Similar conditions were found in Iowa in 1933 by a study of the Brookings Institution, which revealed that if State funds were distributed on the basis of some defensible index such as traffic or vehicle registrations (instead of area) allotments would have been reduced considerably in 75 percent of the counties

In general the conclusion may be drawn that present methods of State fund allocations to local roads and streets are no less heterogeneous and unscientific than are the rates and bases of the taxes through which these funds are raised

ECONOMICS OF USER TAX DISTRIBUTION

The question of what share of State motor vehicle taxes should rightly be allocated to roads and streets other than on the primary system involves fundamental concepts of highway economics The purpose of roadbuilding is to provide for adequate traffic facilities at the lowest possible cost, including both road costs

* Appendix Table A

and vehicle operating costs. In spending for the highway program, therefore, funds must be allocated to those parts of the transportation system where improvements will bring about the greatest reduction in total cost and the greatest utility in adequate service.

Since limited funds do not permit simultaneous betterment of all roads, the element of time is of great moment in an economic distribution of vehicle taxes. If funds were returned to local roads and streets in the amounts generated thereon, prior to adequate development of a system of main highways, the higher cost of transportation for the many vehicles on congested primary routes would far outbalance the reduction in operating costs on the local roads. Also, whereas two road systems may carry equal amounts of traffic, expressed in vehicle miles or gasoline tax receipts, yet the needs of either depend largely on the type and distribution of this travel: whether highway utilization has been intensive, as on heavily trafficked main roads, or extensive, as the dispersed use of a large network of local rural roads. It must also be known in what ratio heavy trucks and buses or pleasure vehicles have accounted for traffic volumes. Furthermore it is important to recognize the integration of motor travel on the various road systems, and the fact that it is the entire trip which must be made at lowest cost, as well as the entire motoring population which must be considered in the computation of total costs for the entire highway system.

The classification of the principal routes as revenue producers is sound in principle. So large a percentage of the actual use of these is recreational in character that the potential increase by reason of wholly adequate facilities should be self-evident. The competitive nature of recreational offerings is also evident. The highways must compete with other classes of recreational inducements. In the business of attracting tourist traffic, route becomes competitive with route, region with region, and even State with State. The impact of the degree of adequacy of major highways has large effects upon both private and public income. The financial support for local road improvements depends to great extent upon the excess earning capacity of the main roads, which in turn is dependent upon the attraction of potential traffic resulting from the offering of satisfactory facilities.

Broader understanding of the purpose of a highway transportation system, viewed as an entity, will demonstrate the importance of such concepts as priority and intensity of use, rather than integrated vehicle mileage alone, as standards by which tax allocations must be measured and financial policies adopted.

TREND IN STATE TAX DISTRIBUTION

Of the total collections of State motor vehicle taxes in 1927, 73.1 percent were used for State highway purposes and 22.0 percent for local roads and streets. By 1936 the percentage of user taxes spent on State roads had decreased to 55.2 percent, while local road allocations increased slightly to 25.1 percent. During this 10-year period, however, total vehicle taxes increased 90 percent, so that the reduced State highway share still represented a 42 percent dollar increase, and the 3.1 percent rise in the local road allotment was an actual 115 percent dollar increase. These figures are shown in Table 1.

In 1927 vehicle funds available for highway purposes were 95.1 percent of the total, whereas last year only 80.3 percent of tax collections were used for highways. This increasing use of road funds for other purposes appears to have hit hardest the State highway systems, though hidden and unreported diversions by local units of government make impossible any definite statement on this subject

There has been more widespread recognition in the past decade of the right of subordinate units of government to share in State taxes For whereas 20 States distributed gasoline taxes to local roads and streets in 1927, in 1936 36 States made such allotments Registration fees were used for local roads by 27 States in 1927 and by 32 States in 1936

TABLE 1

DISTRIBUTION OF MOTOR VEHICLE TAXES FOR HIGHWAYS, 1927-1936*

Year	Total vehicle taxes collected	Amount for State highways	Percent	Amount for local roads and streets	Percent
1927	$ 560,027,983	$409,596,885	73 1	$123,176,360	22 0
1936	1,057,995,000	583,616,000	55 2	265,496,000	25 1
Change 1927-1936	+90%	+42%	−17 9	+115%	+3 1

* Detailed tabulation appears in Appendix, Table B

CITY STREETS

Because funds allotted to counties in many States may be used within municipalities, and because such expenditures are not always reported separately, it has not been possible to determine accurately the amount of State money spent on city streets Accordingly these sums have been included with local road apportionments, and expenditures on urban extensions of State systems have been included in State highway disbursements where it has been possible to segregate them from other local road and street funds The best figure obtainable for State money spent on city streets is $31,468,000, compiled by the U S Bureau of Public Roads for 1936 Eleven States report such expenditures

ADMINISTRATIVE SET-UPS

Highway administrative agencies in the United States include the States, counties, towns and townships, incorporated cities and villages, and miscellaneous local divisions of government In each State the size, type and number of such agencies in operation and the relation or lack of relation among them differ widely

In 4 States all rural roads are administered by the State highway departments, while 26 States * have State and County organizations, 6 have State and township systems, and 12 have three systems State, County, and Township In addition to these rural systems, all States contain municipal organizations which have charge of urban streets, and half the States have further independent or semi-independent divisions within the county, such as commissioners' districts and special assessment districts, both rural and urban

In most States there is neither control by the State over the spending of funds allocated to lesser governmental units, nor is there cooperation between the State and local highway organizations Where laws designate that the State shall approve county construction programs financed with the assistance of State funds such approval is not uniformly followed by adequate supervision of the actual work Where counties are invited to seek the aid and advice of the State, in practice the results are far from reassuring

* Including the State of Washington, although 2 of its counties still contain township units

TREND TOWARD CENTRALIZED ADMINISTRATION

At the close of 1930 there were 324,496 miles of highways under State control By the end of 1936 State controlled mileage had increased to 533,144 miles, a 64 3 percent addition in 6 years Such has been the progress of a movement toward centralized highway administration which began in North Carolina in 1931 By assuming control over the State's 46,800 miles of county roads, North Carolina was the first to consolidate its entire rural highway system under the State highway department

TABLE 2*

TRANSFERS OF LOCAL MILEAGE TO THE STATE HIGHWAY DEPARTMENTS

Year	Number of States	Mileage involved
1931	3	73,651
1932	1	37,028
1933	3	37,744
1934	5	7,190
1935	4	5,623
1936	10	10,696
Total	21†	171,932

* A detailed tabulation appears in the Appendix, Table C

† Several States effected more than one consolidation

It was not long, however, before complete centralization was adopted in West Virginia, Virginia * and Delaware In Maryland 20 out of 23 counties have turned over their roads for maintenance by the State, while a program of consolidation under way in Pennsylvania has resulted in State participation in the maintenance of 46,000 miles of township secondary roads On January 1, 1938, a total of 2,574 miles of Pennsylvania roads in townships, boroughs and cities will be absorbed by the State Popularity of the road consolidation program since 1931 may be judged by figures in Table 2, which show highway transfers to the State highway departments

* Except 3 counties which have elected to retain control of local roads

Twenty-six separate transfers have been made in the six-year period 1931-1936, involving 21 States and nearly 172,000 miles Last year 10 States were involved in such transfers, or twice the number of any previous year

Further consolidations have been effected among the lesser units of government in the assumption of township road responsibilities by county highway organizations It is generally conceded that the township, which in most cases contains an area of 36 square miles or less, has no place in efficient highway administration, and in the past 7 years 4 States have done away with these ineffective highway administrative agencies and adopted a so-called county-unit form of highway organization With this type of administration all roads within the county not a part of the State system are operated as a unit, with locally collected taxes in townships and districts being spent by the central county administration without regard to township or district lines This county unit plan makes possible more economical use of road machinery, a broader tax basis, cooperation and planning, economy in maintenance operations, quantity purchasing, and necessitates the budgeting of funds and the keeping of cost records When Michigan recently completed the transfer of 60,000 miles of township roads to county-unit control, 1,376 small administrative units were eliminated

CAUSES OF CONSOLIDATIONS

The immediate cause leading to centralization of road administration in North Carolina appears to have been the public desire, accentuated by economic depression, to escape from county property tax levies It was proposed that the State assume all future highway financial requirements, with the aid of a one-cent increase in the State gasoline tax, except that the counties should continue payment for the servicing of highway obligations previously incurred The shift of financial responsibility, then, was from property to motor vehicles and from local governments to the State

This centralization plan, however, suggests something more than a temporary relief measure For it is doubtful that the counties would have acceded to such surrender of autonomy had the past record of county highway administration proved efficient and economical That such terms could not be applied to a majority of North Carolina counties was evident from the conditions which the State found upon taking over local road affairs Instead of 67,000 miles of roads listed by the counties only 45,000 miles could be found, despite the fact that 2,590 miles had not been accounted for in the original figure Maintenance varied from satisfactory standards to hopeless inadequacy, and maintenance records in many counties did not exist Some counties were found oversupplied with machinery, others practically destitute, and in nearly all cases machines were either obsolete or badly in need of repair Such causes as these, rather than temporary tax relief, are thought to have been fundamental in the trend toward State assumption of local roads That the trend has not slackened with return to more normal economic conditions may have a bearing upon this point

PROPERTY TAXES FOR ROADS

Whatever is to be said for or against State centralization of highways, the concomitant policy of relieving property of its share in supporting the highway does not conform with the generally accepted theory of highway economics that costs should be paid in accordance with service rendered The shifting of road administration from local to State control involves no alteration in the principle that highways serve other functions than those directly relating to motor vehicles In an equitable allocation of highway costs, rational payments for land service are rightly chargeable to the land which is served Property levies are an essential part of highway income, and their elimination may not only discourage proper development of highway facilities, but may also constitute an unfair burden upon the motorist

A second criticism of policy in connection with highway centralization concerns the tendency of the State to neglect its first responsibility of preserving the integrity of the primary road investment and of providing necessary extensions A shift in administration does not relieve the State of obligations previously assumed, and the requirements of the main road system must be recognized prior to further tax allocations

A large element of overriding the recommendations and warnings of the State highway departments has characterized the adoption of State policies throwing the cost burden of additional large mileages upon the incomes from user taxes available to the department and usually inadequate for the requirements of the existing major highway systems

CRITICISM OF SMALL ADMINISTRATIVE UNITS

It is self-apparent that many small roadbuilding entities now in operation are outworn relics of the dependence of transportation upon the horse that both the time and distance of travel upon which their limits were fashioned have been reduced to negligible importance Administrative scope has expanded, and this fact must be recognized by eliminating the multiplicity of highway organizations of minor units of government which make impossible the operation of highways as a coordinated system A small unit is generally unable to afford proper engineering personnel, its staff may be subject to frequent changes because of elections, and in general undue emphasis is likely to be placed upon political rather than technical considerations Short radii of operation make the use of modern road machinery uneconomical through excessive overhead and numerous duplications, while small purchases of supplies and materials impose penalties of higher unit prices Variations among the jurisdictions in area, population, taxable valuation, road mileage, topography, climate, vehicles registered and traffic volumes may make possible the extension of road facilities beyond traffic requirements in one county, while a neighboring unit may be financially unable to provide the taxpayer with a lasting return for the money he pays for satisfactory highway services Budgeting, accounting, debt control and planning are generally beyond the pale of local road administration, while lack of continuous maintenance, the use of force account methods, and incompetently controlled spending of funds collected outside the local jurisdiction are weaknesses generally in evidence

VARIATIONS AMONG COUNTIES AND STATES

In most discussions relating to the merits or demerits of centralized government it is claimed on the one hand that the county is " too small " to effect a proper highway administration, and on the other that the State is " too large " Either statement implies that counties and States are essentially homogeneous, and that there exists a standard-size government unit most applicable to proper highway management Yet neither counties nor States are homogeneous units Counties may differ in area from the 25 square miles of Arlington County, Virginia, to San Bernardino's 20,175 square miles in California This latter county is larger than the three States of New Jersey, Delaware and Maryland combined In population variations are even more pronounced, Loving County, California, for example, having but 195 residents compared with 4 million persons living in Cook County, Illinois As regards the States, the largest is 250 times the area of the smallest, while populations vary in the ratio of 138 to 1 Nine States have more than 100,000 miles of highways (Texas has over 200,000) while six have less than 15,000 The fact that a county may be larger than the State of Delaware, in which State centralization of highways is in effect, presents the possibility that the State may actually be "too small" and the county "too large "

Consideration of the county as a highway administrative unit must take into account the two different general types of county, the rural, and the urban The rural county is often unadapted to the performance of highway functions because of the limitations of its resources and the lack of sufficient highway activity to permit large-scale operations, either intensive or extensive The urban county which contains a large city and considerable traffic and population, however, is by reason of its wealth, responsibilities, and intensive road needs, a logical highway administrative unit Such urban counties nevertheless are handicapped in their function of improving highways by the fact that they are usually part of a larger metropolitan area embracing more than one county, as well as lesser jurisdictions such as towns and villages Definite legislation is accordingly needed for effectuating correlated action throughout the metropolitan district, both in planning the transportation system as a whole and in detail, and in fixing priorities for the improvement program It is necessary, therefore, to distinguish between such counties, and to recognize that to speak merely of the size of an administrative unit may be inconsequential, if not misleading.

Since such special considerations must be taken into account, it seems obvious that no definite standard-size unit can be prescribed which will be universally absolute for highway administration. The intensity of highway needs varies, as well as the degree to which a region has been developed and the type of its development. Large agricultural regions might prove nearer the optimum unit for highway administration than large areas of concentrated industrial development. Physical characteristics such as topography and climate are important factors for consideration as well as possible sources of highway funds and probable necessary expenditures.

THE OPTIMUM SIZE OF HIGHWAY UNITS

Certain characteristics of local government mentioned are susceptible to correction, such as lack of planning, budgeting, and other administrative matters. It is claimed by the opponents of centralization that county government may be revived by effecting reform along these lines. But many criticisms against the local highway unit as an administrative body are functions of physical characteristics which are not susceptible to "reform." No matter how efficient its system of accounting nor how expert its highway commission, local government may still be limited to uneconomical operations unless it is able to raise sufficient funds to pay the highway bill and unless the scope of construction and maintenance requirements will allow fullest utilization of equipment, a proper distribution of overhead and the economical operation of a competent engineering organization.

The economist recognizes that a profitable industrial plant is limited in its physical equipment to an optimum unit of operation; that unwieldy production units cause economies of large-scale production to give way to dis-economies, and that particular circumstances may alter the optimum plant even in the case of similar products. On the other hand, horizontal combination of a number of optimum production units under centralized administration is entirely in keeping with economical operation. The so-called American trust is an example of such horizontal combines. In other words an industry may require technical decentralization and managerial centralization.

This principle of economics appears to be applicable to the provision of highway facilities, in which optimum highway operating units might be determined upon, and their management directed centrally. Such is the general plan adhered to in the division of State highway systems into engineering districts, and suggested in the relation existing between the Federal and State governments.

It does not appear unworkable that all rural roads in a State might be operated on a similar basis. Each State might contain several highway operating units varying as to optimum sizes in accordance with particular considerations. These districts might be a grouping of counties or other local jurisdictions into regional areas. In small States or States essentially agricultural the entire area might be determined the optimum, in which case consolidation of all roads in the State would be economically in order. Whatever the size and number of operating units, however, financial and planning administration might still be centered in the State.

The establishment of the State highway departments was recognition of the need of centralized administration in creating a primary system of roads, and in the spending of State vehicle taxes with wisdom and coordination for the best interests of the whole State. Local units of government on the other hand were left to administer their individual highway affairs, which were truly local affairs financed by local money With the State-wide extension of motor transport, however, all roads within a State developed into a network which it was necessary to view as a whole Recognition of the wider influence of secondary roads was granted in the form of allocations of State money to local units of government which were not established to be expending agencies for such funds Accordingly, the principle came to be tolerated that there should be centralization of certain highways in the State, financed by State funds, and decentralization of certain other roads, also financed with State taxes, in a multiplicity of lesser governmental units There is basic conflict between these two policies On the one hand it is accepted that the highways constitute a closely-knit system, on the other hand uncorrelated policies of finance develop them as a patchwork

The chief objections to State control of all highways are for the most part political rather than economic That is, there is general recognition of the possibilities of economy and coordination with control centered in the State highway department, but there is fear concerning the effect on local government which might result from eliminating local highway administration Such action, it is asserted, would tend to discourage interest in other local governmental functions and eventually to bring about complete State centralization This would be the first step, according to stock arguments, toward the destruction of self-government, individual initiative, and democracy

The "fine-woven rhetorical expressions" advanced in behalf of local government, it is pointed out, must be tempered with the common-sense observation that highway transportation is not a function properly confined to imaginary and outmoded political boundaries To claim that the preservation of democracy depends upon the maintenance of such a system has been construed by some as an argument for governmental waste and inefficiency, and to extol the small local unit as a "school for democracy" has been challenged on the grounds that accounting and engineering are so often omitted from its course of study The statement has been made that if democracy can coexist with such philosophies of government there is little fear that it would perish from State financial administration of highways

FACTORS SUPPORTING CENTRALIZATION TREND

A consideration of importance with regard to the future possibilities of centralized highway administration is the recently inaugurated Federal assistance for secondary road development During the depression years secondary roads and urban streets were granted various emergency appropriations by the Federal Government for the prime purpose of furthering employment In the present fiscal year, however, regular Federal aid grants of $25,000,000 are available for secondary road improvement, to be matched by equal amounts of State funds It is of significance that the State highway departments may employ the services of competent county highway organizations, acting under direction of the State, in the preparation of plans, surveys and specifications, and in the supervision of construction Where laws limit the State highway department in the extent of mileage it can maintain, the State may draw up agreements with lesser governmental units which will attend to the maintenance of these secondary roads No such agreement will be approved, however, if any road previously built with Federal funds and currently maintained by a county or lesser political unit is not being kept in satisfactory condition

CENTRALIZATION AND PLANNING

A further development toward closer cooperation between State and county, and greater control by the State over local roads is the promising possibility of State-wide highway planning. Surveys now under way to provide the facts necessary for plans may be made the instrument for publicizing the inadequacies of small highway units, and for revealing to the taxpayer how much of his money supports obsolete governmental machinery instead of better roads. It is also hoped that State legislation may follow the findings of such surveys when questions of highway administrative reform arise.

Some of the immediate purposes of the State-wide planning surveys are:

1. To define the mileage of roads within each State to be supported by public funds.
2. To determine the use made of the parts of this system, hence the sources of necessary taxes and their proper distribution.
3. To determine future construction requirements for extensions, improvements and replacements.
4. To determine the priority of such construction projects.
5. To estimate necessary maintenance operations.
6. To estimate future highway income and to budget this sum according to estimated future financial requirements.

These several purposes emphasize the need for control by a central agency to supersede uncoordinated plans which result from the operation of a large number of highway jurisdictions acting independently. In order that planning may be effective throughout the State there must be an administrative control with greater power than any of the separate minor units. Planning which is "State wide" cannot be attained by a number of individual plans within the State, but only by a central plan which applies to an integrated system.

In review of the foregoing status and trends in State vehicle tax distribution for highways and in highway administrative procedure, a summary of the data is presented, followed by a list of conclusions and recommendations suggested by them.

A SUMMARY OF FACTS

1. Approximately one-fourth of all State motor vehicle taxes were distributed for local road and street purposes in 1936.

2. The share of State funds allocated to local roads and streets has increased only 3.1 percent in the last ten years, while the actual money so distributed shows a 115 percent dollar increase during the same period.

3. The State highway share of motor vehicle taxes has decreased more than 17 percent in 10 years, while the dollar allotment has increased 42 percent.

4. State funds are distributed to local units of government in widely varying amounts and without regard to traffic generated, five States making no allocations and one distributing more than 24 million dollars.

5. Methods of distribution among each separate local unit are generally untenable, being made in equal amounts or on the basis of area, population, road mileage, assessed valuation, vehicle registrations, tax collections, or a combination of two or three of these.

6. In most cases the States retain no control, or merely nominal control, over the spending of vehicle taxes used on local roads and streets.

7. Four States have consolidated all rural roads in the State highway departments, while 26 States have State and county organizations, 6 have State and township units, and 12 have three systems: State, county and township.

8 In the past 6 years 21 States have shifted 171,932 miles of local roads to State control, constituting a 64 3 percent increase in State mileage during that period

9 More States were involved in local road consolidations in 1936 than in any previous year

10 In the past 7 years 4 States have eliminated all township road units

11 The highway consolidation movement has shifted the highway tax from local to State government and from property to motor vehicles

CONCLUSIONS AND RECOMMENDATIONS

1 Allocation of State vehicle taxes to local roads and streets should be made with reference to both volume and intensity of traffic generated, but with consideration for the priority of primary road requirements so that transportation facilities for the integrated system may be adequate and at lowest total cost

2 The State should maintain adequate control over all projects on which State money is used

3 Arbitrary political boundaries have no relation to functions of highway transport

4 A highway operating unit may be limited in its ability to function economically by reason of certain characteristics inherent in small-scale operations

5 A highway administrative area is not necessarily limited to the optimum unit determined upon for construction and maintenance operations, and should embrace sufficient area to permit quantity purchasing, specialized personnel, and a coordinated highway program

6 With the transfer of local roads to State control, benefits to land remain a legitimate highway service which should be recognized by property contributions to the highway fund

7 It is important that the State should provide first for all primary road obligations before assuming added burdens in connection with local roads

8 Federal aid for secondary roads is recognition of the fact that such parts of the highway system give more than local service This new Federal policy promises to create closer cooperation between States and local units

9 State-wide planning surveys constitute the first wholesale attempt to bring before the public and legislative bodies facts concerning the need for sane financial and administrative policies

10 State-wide plans cannot be successful without a central planning authority

11 The failure of any State to provide a major system of highways not only adequate but attractive to the rapidly growing tourist and recreational traffic results in large losses of potential income to the public from the user taxes and to private business relying upon the highway travel

12 The failure to establish and to follow sound principles of financial administration is a serious cause of lack of progress toward adequate major highways where this condition exists

13. The waste of highway funds by duplicate local units and the uneconomical operations they necessitate brands financial administration the least progressive field of highway transportation

APPENDIX

TABLE A*

Legal Provisions Regulating the Use of State Motor Vehicle Funds for Local Roads and Streets

1—Gasoline Taxes

State	Tax rate (Cents)	Distribution to local roads and city streets
Alabama	6	3 cents to counties, distributed equally
Arizona	5	3/10 to counties, according to gasoline sales in each
Arkansas	6 5	7 7 percent to counties, on basis of population, registration and area
California	3	1/2 to counties $5,000 for each county and county-city, four times per year Balance distributed according to registrations
Colorado	4	27 percent to counties, 3 percent for extensions of State system in cities, towns and counties, on basis of State mileage in counties
Connecticut	3	
Delaware	4	
Florida	7	3 cents to counties, distributed among them by particular statutes
Georgia	6	1 cent to counties on basis of State-aid mileage in each
Idaho	5	
Illinois	5	1/3 to counties, 1/3 to municipalities, on basis of vehicles registered
Indiana	4	40 percent to counties, 10 percent to cities, according to population
Iowa	3	4/9 to counties, by area
Kansas	3	
Kentucky	5	
Louisiana	5	
Maine	4	To general highway fund, with registration fees, from which $150,000 goes to town roads, $700,000 to 3rd class roads, on mileage basis, and $1,000,000 to State-aid roads according to town valuation
Maryland	4	1 05 cents to counties, by mileage of county roads, 1 15 cents to Baltimore city
Massachusetts	3	
Michigan	3	To State highway fund, with registration fees, from which $6,000,000 goes to counties, 7/8 in proportion to fees collected, 1/8 equally
Minnesota	3	1/3 to counties, based on mileage and traffic needs
Mississippi	6	2 1/2 cents to counties, on basis of population, registrations and area
Missouri	2	
Montana	5	
Nebraska	4	3/8 to counties
Nevada	4	
New Hampshire	4	Small amount to some local roads (less than 9 percent of total in 1936)
New Jersey	3	$5,000,000 to city streets
New Mexico	5	
New York	3	5 percent to New York City, 20 percent to counties, by mileage
North Carolina	6	
North Dakota	3	1/3 to counties on basis of registration fees collected
Ohio	4	3 cents, minus about $285,000 to counties, villages and townships on basis of vehicles registered
Oklahoma	4	1/4 to counties, according to population and area
Oregon	5	
Pennsylvania	4	1/2 cent to counties, based on gas tax returns during preceding 3 years
Rhode Island	2	
South Carolina	6	1 cent to counties, based on registrations
South Dakota	4	

* Data incomplete

TABLE A—*Continued*

State	Tax rate (Cents)	Distribution to local roads and city streets
Tennessee	7	To counties 1 cent equally, ½ cent by population, and ½ cent by area
Texas	4	
Utah	4	
Vermont	4	$500,000 to local roads, by mileage
Virginia	5	$239,000 in 1936 for the 3 counties not under State control
Washington	5	3 cents to counties and cities, according to gas sales
West Virginia	4	
Wisconsin	4	
Wyoming	4	25 percent to counties, based 30 percent on area, 30 percent on rural population, and 40 percent on assessed valuation

TABLE A—*Continued*

2—Registration Fees

State	Distribution to local roads and city streets
Alabama	20 percent to incorporated municipality or county where owner resides.
Arizona	50 cents of original fee retained by county.
Arkansas	
California	Approximately 30 percent to counties in proportion to registrations.
Colorado	50 percent to counties in proportion to collections.
Connecticut	
Delaware	
Florida	
Georgia	
Idaho	90 percent retained by counties.
Illinois	
Indiana	¼ to counties and cities; counties, ⅞ on mileage, ⅛ on population; cities, on basis of population.
Iowa	
Kansas	10 cents of each registration to county.
Kentucky	
Louisiana	
Maine	
Maryland	After debt service and operating expenses of motor vehicle department, traffic court, etc., 30 percent to Baltimore.
Massachusetts	
Michigan	See gas tax data.
Minnesota	
Mississippi	All to counties where collected.
Missouri	
Montana	All to counties where collected
Nebraska	5 cents retained by counties for each original registration.
Nevada	
New Hampshire	Small sum for State-aid ($272,000 in 1936).
New Jersey	Carrier taxes to municipalities.
New Mexico	15 percent to counties in proportion to registrations.
New York	25 percent to counties.
North Carolina	
North Dakota	
Ohio	47 percent to counties where car registered.
Oklahoma	9 percent to cities, 51 percent to counties.
Oregon	
Pennsylvania	
Rhode Island	
South Carolina	
South Dakota	76.5 percent to counties where collected.
Tennessee	
Texas	100 percent to county where collected, up to $50,000; 50 percent up to $175,000.
Utah	
Vermont	
Virginia	
Washington	
West Virginia	
Wisconsin	20 percent retained by town, village and city; also $3,000,000 to counties for State-aid roads, 40 percent on basis of registrations and 60 percent by mileage.
Wyoming	County registration fees retained.

TABLE B

Disposition of State Motor-Vehicle Receipts

To State Highways and Local Roads and Streets 1927-36

	Total funds distributed	For State highway purposes	Percent	For local roads and streets	Percent	Total fund to State highways, local roads and streets	Percent
1—Registration Fees							
1927..	$ 301,061,132	$ 220,645,359	73.3	$ 61,543,245	20.4	$ 282,188,604	93.7
1928..	322,630,025	235,142,906	72.9	66,569,311	20.6	301,712,217	93.5
1929..	347,843,543	250,704,624	72.1	73,226,339	21.1	323,930,963	93.2
1930..	355,704,860	253,013,603	71.1	74,639,463	21.7	327,653,066	92.8
1931..	344,337,654	234,593,379	68.1	79,388,101	23.1	313,981,480	91.2
1932..	324,273,510	188,539,140	58.1	83,298,207	25.7	271,837,347	83.8
1933..	301,315,447	157,754,844	52.4	75,943,682	25.2	233,698,526	77.6
1934..	318,576,965	175,382,722	55.1	84,356,966	26.5	259,739,688	81.6
1935..	324,855,135	173,477,594	53.4	87,587,250	27.0	261,064,844	80.4
1936..	374,921,000	194,491,000	51.9	98,241,000	26.2	292,732,000	78.1
Total	$3,315,519,271	$2,083,745,171	62.8	$ 784,793,564	23.7	$2,868,538,735	86.5
2—Gasoline Taxes							
1927..	$ 258,966,851	$ 188,951,526	72.9	$ 61,633,115	23.8	$ 250,584,641	96.7
1928..	305,233,842	225,315,715	73.8	68,562,491	22.4	293,878,206	96.2
1929..	431,636,454	318,087,598	73.7	101,961,887	23.6	420,049,485	97.3
1930..	494,683,410	359,797,465	72.7	118,247,702	23.9	478,045,167	96.6
1931..	537,589,717	381,711,610	71.0	134,318,053	25.0	516,029,663	96.0
1932..	514,138,900	336,144,197	65.4	127,220,400	24.7	463,364,597	90.1
1933..	519,403,450	314,432,266	60.5	153,777,094	29.6	468,209,360	90.1
1934..	565,139,596	333,196,930	59.0	138,338,782	22.5	471,535,712	81.5
1935..	615,580,975	348,651,966	56.6	150,546,567	24.5	499,198,533	81.1
1936..	683,074,000	389,125,000	57.0	167,255,000	24.5	556,380,000	81.5
Total	$4,925,447,195	$3,195,414,273	64.9	$1,221,861,091	24.8	$4,417,275,364	89.7
3—Total Motor Vehicle Taxes							
1927..	$ 560,027,983	$ 409,596,885	73.1	$ 123,176,360	22.0	$ 542,773,245	95.1
1928..	627,863,867	460,458,621	73.3	135,131,802	21.5	595,590,423	94.8
1929..	779,479,997	568,792,222	73.0	175,188,226	22.5	743,980,448	95.5
1930..	850,388,270	612,811,068	72.1	192,887,165	22.7	805,698,233	94.8
1931..	881,927,371	616,304,989	69.9	213,706,154	24.2	830,011,143	94.1
1932..	838,412,410	524,683,337	62.6	210,518,607	25.1	735,201,944	87.7
1933..	820,718,897	472,187,110	57.5	229,720,776	28.0	701,907,886	85.5
1934..	883,716,561	508,579,652	57.5	222,695,748	25.2	731,275,400	82.7
1935..	940,436,110	522,129,560	55.5	238,133,817	25.3	760,263,377	80.8
1936..	1,057,995,000	583,616,000	55.2	265,496,000	25.1	849,112,000	80.3
Total	$8,240,966,466	$5,279,159,444	64.1	$2,006,654,655	24.3	$7,285,814,099	88.4

TABLE C
ROAD CONSOLIDATIONS

Year	State	Local road mileage transferred to State
1931	North Carolina	46,826
	Pennsylvania	20,167
	Louisiana	6,658
	Total	73,651
1932	Virginia	37,028
	Total	37,028
1933	West Virginia	29,098
	Oregon	2,046
	California	6,600
	Total	37,744
1934	Minnesota	4,356
	Missouri	937
	Georgia	367
	Indiana	871
	Kentucky	659
	Total	7,190
1935	Delaware	2,602
	Nebraska	1,391
	Missouri	834
	Nevada	796
	Total	5,623
1936	Arizona	428
	Georgia	648
	Kentucky	340
	New Mexico	2,021
	Ohio	2,391
	Oklahoma	606
	South Carolina	419
	Texas	579
	Missouri	914
	Pennsylvania	2,350
	Total	10,696
Total Transfers, 1927-36		171,932

REPORT OF DEPARTMENT OF HIGHWAY TRANSPORTATION ECONOMICS

R L MORRISON, *Chairman*

REPORT OF COMMITTEE ON UNIFORM HIGHWAY ACCOUNTING

PROGRESS REPORT ON DEVELOPMENT OF A SYSTEM

BY ANSON MARSTON
Dean of Engineering (Emeritus), Iowa State College
AND
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SYNOPSIS

A previous progress report to November 1936 will be found in the Proceedings, Highway Research Board, Vol 16, p 46 Since that date, the Committee on Uniform Accounting of the American Association of State Highway Officials has proceeded very actively and successfully with its work First, 8 regional conferences were held to contact the maintenance engineers and the chief accountants of the several States Then a special subcommittee was appointed, consisting of one member at large and 8 regional representatives each of whom proceeded to visit and confer with each State highway department in his region Upon completion of these visits, the subcommittee met at Ames, Iowa, June 22-25, 1937, agreed upon the general features of a system of uniform highway accounts, and assigned the preparation of tentative manuscript sections thereof to a writing committee of four This writing committee met at Washington, D C, September 20-26, 1937, and completed a report which was presented, discussed, and adopted September 27-30, at the Boston meeting of the American Association of State Highway Officials by both the Association's full Committee on Uniform Accounting and by its Committee on Administration The Maintenance Committee concurred The Executive Committee has submitted the recommendations to the States for letter ballot

Included in the report thus adopted is a system of highway maintenance cost accounts, it is proposed that this shall constitute one of the sections in the final Uniform Highway Accounting Manual

Also included in the adopted report is a plan for active further work during 1937-1938 For this work, the United States has been divided into ten regions, in which ten regional conferences are being held for the study of construction cost classifications, equipment costs administrative expense, and fixed capital, income, balance sheet and general ledger accounts, including depreciation accounts for all fixed-capital property

When finished, the manual will contain classifications and suggestions for setting up and operating a system of double entry uniform highway accounts, classified and numbered, fully comparable with the uniform systems of accounts for steam railroads and other utilities prescribed by the U S Interstate Commerce Commission, the Federal Power Commission, and other regulatory authorities

The United States is now rapidly accepting the concepts that the highways are of the nature of public transportation utilities, and that the users thereof should pay for the services rendered and benefits received in proportion to their value, in consequence, it is becoming increasingly apparent that the State Highway Departments must have a uniform accounting system that will enable each State to determine quickly and accurately what it is costing to produce and maintain its highway transportation services. Further, motor vehicle transportation is becoming more and more a national industry, unlimited by State boundaries, it is destined to become still more national in character. The records of the various States are being compared as to construction cost, maintenance cost, and highway user tariffs. Comparisons of these figures are rendering individual States gross injustices because the layman, the legislator, and the news reporter do not know that the reports showing differences in costs in different States are not compiled upon the same basis, the operation costs in two States may appear to differ widely although actually in close agreement.

In view of these new concepts regarding the functions and responsibilities of highway departments, the States, through their Association of State Highway Officials, are actively developing a system of uniform classification of highway accounting, which it is hoped will ultimately become the standard for all States in the same way that the railroads, the electric utilities, and many other organizations of similar character and purposes have reached common standards of accounting. To further this work, a committee of the Association has been given the responsibility of formulating the recommendations. A project committee of the Department of Highway Transportation Economics of the Highway Research Board was appointed last year to work with the Committee of the American Association of State Highway Officials, and to give special attention to the accounting classifications which are needed in studies of the economics and costs of highway transportation. The past and present work of the Department of Highway Transportation Economics has been very much handicapped for lack of reliable highway investment figures and maintenance costs.

The progress made during the past twelve months is highly gratifying. Everyone is encouraged to expect the realization of a difficult goal much sooner than was anticipated a year ago. The following review of the work of the Association committee during the past year and of its plans for the coming year will indicate the progress.

The Association's first committee on accounting was organized in 1926. Fundamental principles of accounting and accounting organization in the several States were the topics of discussion of this committee until 1929, at which time its report on "Standardized Definitions of Highway Terms and Highway Cost Analyses" was submitted. The committee was not active again until 1936, when it was reorganized for the purpose of extending the work of the early committee to include actual classification of accounts on a uniform basis, and the preparation of a manual of instructions and suggestions. The reorganization of the committee and revivification of the work were prompted partly by a resolution of the Mississippi Valley Highway Association in February 1936, petitioning the U. S. Bureau of Public Roads to devise and establish a uniform system of accounting, and partly by repeated suggestions from the Department of Highway Transportation Economics of the Highway Research Board that something be done to make available really comparable cost reports from State highway departments which would furnish the figures necessary to the studies of transportation economics desired by the various department committees.

Under the auspices of the Association committee on accounting, eight regional meetings were held in November 1936 and January 1937, for the purpose of discussing with representatives of the State highway departments the classifications and procedures which the committee should consider These meetings were attended by 90 State accountants and engineers, representing 38 States The opinions offered at these meetings were unanimous in expressing the belief that there was immediate need for uniform accounting practices, and that the States would cooperate to the extent permitted by State laws by modifying present practices to conform to a uniform basis once such a system was adopted by the association.

To make such cooperation immediately active and effective, a special subcommittee was appointed, consisting of one member at large and 8 regional representatives, each of whom proceeded to visit and confer with each State Highway Department in his region Upon completion of these visits, the subcommittee met at Ames, Iowa, June 22-25, 1937, agreed upon the general features of a system of uniform highway accounts, and assigned preparation of tentative manuscript sections of a manual to a writing committee of four This writing committee met in Washington, D C, September 20-26, 1937, and prepared the committee's report to the Annual Meeting of the American Association of State Highway Officials, Boston, September 26-30, 1937 A general classification of uniform highway accounts was tentatively recommended, as shown in Appendix I to this paper Because maintenance cost accounting seemed to be the item on which interest most centered at the regional meetings, the committee undertook a special study of it and submitted at the Boston annual meeting a recommended uniform classification of maintenance accounts, together with suggestions for its use These recommendations were concurred in by the Maintenance Committee of the Association, were adopted by the Committee on Administration, by the Executive Committee and have been now submitted to the States for a letter ballot The recommended definition of maintenance and the recommended classification of maintenance accounts are given in Appendix II to this paper [1], at present there is much conflict of practice as to the dividing line between maintenance and construction

Some preliminary consideration was given by the committee to fiscal accounts and to administration, construction, and equipment cost accounting Thorough discussion of these items is planned in a new series of ten regional meetings with State representatives, similar to the eight that were held last year Four of these meetings have been held, three are scheduled during this month, and the other three will have been completed by March 4

Following these meetings, the committee expects to prepare definite recommendations to be submitted to the 1938 annual meeting Thus, section by section, the committee is planning to cover the entire scope of highway department accounting Ultimately, the recommendations, classifications, and suggestions will be coordinated in a complete manual on highway accounting [2]

The general character of the proposed Uniform Highway Accounting Manual is shown by the following quotation from material prepared by G H Lloyd, Chairman of the American Association of State Highway Officials' Committee on Uniform Accounting, for discussion in the 10 regional conferences now being held

"The accounting system, to be understood and followed uniformly and to prevent disin-

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[1] The numbers attached to the several accounts will be changed in the final manual

[2] It should be emphasized that it is not the intention of the committee to recommend the details of procedure of organization, office practice, accounting equipment, and other items which are felt to be State rights

tegration, should be placed in manual form. A uniform manual for all State highway departments, providing for standardization of accounts and definitions, should be of great assistance to the managements of the departments in supplying comparative costs on work accomplished. This Uniform Accounting Manual could follow along the same lines as the Uniform Accounting Manual used by railroads. Basic accounts should be included and to some extent suggestions for methods of handling the detailed source preparation of the accounting information. It is recognized that there must be some variance in each of the States due to statutory limitations and to the highway department organization; however, most of the highway department organizations are very similar and it is felt that the Uniform Accounting Manual, by laying down general principles and outlining the source preparation of accounting information, could be adapted to each type of State highway organization and fit each statutory limitation.

"The Uniform Accounting Manual should provide for a chart of fiscal accounts with an explanation of the purpose and use of each. It should also provide for a uniform chart of operating cost accounts for Construction and Maintenance. These charts of accounts should not be in such detail as to handicap any State, but should provide for more general classifications, and each account should be properly explained. Enlargement of the accounts and provisions for more detail could be added by the States, depending upon their individual needs or requirements. The arrangement of the accounts should be in such manner as could be used by a department whether operating on a centralized or decentralized type of organization.

"The Uniform Accounting Manual should not be designed to fit any type of accounting machine nor attempt to develop any detail forms, procedure or accounts requiring the use of any special type of office equipment or accounting machines. Any accounting system would unquestionably be more efficient and give better satisfaction, however, if modern accounting equipment were used in connection with the system. Modernized equipment is as necessary to the success of an accounting system as modernized equipment is to the successful and economical maintenance and construction of highways.

"It must be borne in mind that your highway costs will be no better than your system provides, and since the source material of highway costs is supplied by foremen, superintendents, etc. who are usually not trained in the accounting field, there should be an accounting manual in each individual State to fit its particular needs. This manual should take up where the Uniform Accounting Manual leaves off, and should provide for detailed instructions in permanent form. These detailed instructions in the State manual should be strictly complied with in order that the system might function smoothly. The manual should be concise and presented in a simple and plain fashion to be easily understood by any person, and should cover the following material:

1. List of all forms to be used and instruction for the use of each form.
2. Detailed explanation of pay rolls and their preparation.
3. Detailed explanation of routine, internal checks, responsibilities, etc.
4. Rules, regulations and methods of handling depreciation, equipment operation costs, stores and administration.
5. A complete detail chart of accounts showing the symbols, description and purpose.
6. Reports to be prepared, giving complete description as to what is included and for what purpose they serve.

"Where changes are made in the accounting system of a State highway department, it is usually advisable not to attempt to make a complete change at one time. When there is a complete turnover of procedure and accounts, confusion usually exists among the employees handling accounting work which might cause discouragements, wrong impressions, and possibly ultimate inefficiency in the system. Thorough study should be given to the old system and old records, as many such old records could be used very well under the new system and should serve a very valuable purpose for comparisons and assistance in maintaining continuity in records. As sections of the new system are initiated by a State highway department, the employees participating in the preparation of source material and routine handling should be thoroughly schooled in the instructions and requirements before the new work or change is undertaken.

"In this connection it is pointed out that the installation of an accounting system does not consist in simply providing forms and procedure. The personnel having connection with the details as provided in a State accounting manual should be thoroughly trained and should clearly understand their functions to secure success of the system provided. They should understand also the necessity for complying strictly with the instructions in the manual.

"By way of summarization, a Uniform Accounting Manual for State highway departments should incorporate definitions, standardized account classifications for fiscal and cost accounts, with explanations, and only such general procedure for preparation of source material as necessary to secure uniformity of information under the standardized accounts, leaving all detail forms, procedure and accounts to the individual States which would be developed under separate State accounting manuals"

THE ESSENTIAL FEATURES OF SATISFACTORY UNIFORM HIGHWAY ACCOUNTING

The authors wish next to discuss the essential features of what they at present would consider a satisfactory system of uniform highway accounting

In the same document quoted just above, G H Lloyd, Chairman of the Committee on Uniform Accounts, has said

"The need for a well designed and comprehensive accounting system is apparent to the executives of every State highway department It is essential that the system be designed to furnish complete information on all departmental operations and that it be sufficiently flexible to readily adjust itself to any change in organization, policies or laws The system should embrace both *Fiscal* or *Fund Accounting* and *Cost Accounting*

"*Fiscal Accounting* deals with the payment of bills, pay rolls, contractors' estimates, collection of income or receipts, and the handling of all necessary journal entries that affect the General Ledger and the various income and expenditure accounts of the department This work includes the recording and handling of all fund accounts; appropriation accounts, and classification of expenditures, and provides control accounts to insure the accuracy of the cost accounting records Fiscal Accounting information should be collected according to functions, classes of work, organization units, and the object or purpose of expenditures

"*Cost Accounting* should include the collection of costs and statistics which have to do with the measurements of results accomplished against money spent, or the cost of the operating units of work The cost accounting work of the highway department should be separated from the fiscal accounting work, but it is highly important that the cost accounting records balance with the controls established in the fiscal accounting ledgers The classification of accounts to be kept under the cost accounting section are for the purposes of accumulating costs into operating accounts (Maintenance and Construction), so that the executives and responsible department heads may review the efficiency of performing the work of the department"

Mr E E Hall, Secretary of the American Association of State Highway Officials' Committee on Uniform Accounting, has said [3]

"The building, maintenance, and administration of State highways is "big business" Even in the State having the smallest expenditures in 1936 it cannot be considered a small business, for it was well beyond the three million class, while the State having the largest expenditures made payments in excess of 80 million dollars The business of a number of highway departments exceeded 25 million dollars last year (It may be added by the authors that in the entire United States over $1,100,000,000 was spent on State highway systems alone in 1936, and that the total expenditures on State systems since the World War has been about $13,000,000,000

"It should be apparent that if commercial big businesses maintain complete fiscal and cost accounting records, then considered from magnitude only, the State highway departments should have equally adequate systems Actually, the State highway departments, because of their public character, have more cause to maintain complete records of accounts, and be in a position to prepare financial statements as comprehensive and complete as any big business"

"Believing that our State highway departments should be considered as the operating companies of a vast national system of highways, the fundamental principles of accounting should be predicated on needs of a transportation system, and not merely on the basis of recording expenditures of appropriated funds Further the basic accounting classifications should be essentially the same for all 48 operating States"

Mr Hall further outlined his views on highway account as follows

"The following general classification of accounts is suggested as a basis of discussion and

[3] In material prepared by the American Association of State Highway Officials' Committee on Uniform Accounting for discussion at the 10 regional meetings now being held

as a foundation upon which the accounting system may be developed

Current Assets
- Proprietary accounts
  - Cash
  - Accounts receivable
  - Inventories
  - Prepaid expense
- Budgetary accounts
  - Estimated revenues
  - Unappropriated authorization

Fixed Assets
- Land, other than R/W
- Buildings
- Less reserve for depreciation
- Investment in highways
- Less reserve for depreciation
- Work under construction
- Equipment
- Less reserve for depreciation

Expense
- By detailed classifications

Current Liabilities
- Proprietary accounts
  - Accounts payable
  - Short term notes payable
  - Accrued interest
- Budgetary accounts
  - Encumbrances
  - Appropriations

CURRENT SURPLUS

Deferred Liabilities
- Bonds outstanding
- Other long term obligations

Reserves

CAPITAL SURPLUS

Income
- By detailed classifications

"It is suggested that commercial accounting practice be followed very closely in so far as a double entry set of books kept on an accrual basis is concerned It is also desirable that the general ledger contain either in summary or in detail the record of all financial transactions, supported by the necessary subsidiary ledgers and records However, the absence of accounts and procedures concerned in determining profit and loss and individual ownership, and the use of budgetary accounts are characteristic of State highway department accounting

"It will be observed that the suggested classification completely segregates current assets, current liabilities, and operations, including income and expenses, from fixed assets and deferred liabilities This actually results in a *dual balance sheet,* the difference between the current assets and current liabilities being represented by a *current surplus,* while the difference between fixed assets and deferred liabilities is represented by *capital surplus* These are in lieu of the ordinary profit and loss, and investment accounts of commercial concerns"

Appendix I illustrates the *general ledger accounts* referred to by Mr Hall The general ledger would show the balances from the various subsidiary ledgers in all the accounts listed

Appendix II illustrates the primary classification of accounts which will be required for the maintenance cost section of a satisfactory system of uniform highway accounts

THE AUTHOR'S VIEWS ON UNIFORM HIGHWAY ACCOUNTING

The State highways of each State constitute a great public utility,[4] owned and maintained by the State for the purpose of supplying road services of various kinds to the public In return, those using these services are required to pay for them in various ways For *road access* services, the real estate owners served pay property highway taxes, for *road use services,* travelers on the highways pay motor-vehicle licenses fees, motor fuel taxes and ton-mile taxes In reality, none of these payments are true *taxes,* they all are *charges for road services supplied;* just as payment of bills for electric light and power are payments for electricity furnished This public utility character of highway systems is one of the main facts which should be recognized and understood in developing a uniform highway accounting system Even more than in the case of privately owned utilities it is essential that highway accounts supply true full detailed, clearly arranged and summarized information concerning

First, all of the State's property now used for highway purposes, its cost new, its *present accrued depreciation* and its *present value*

[4] See Anson Marston, "Valuation of Highways," Highway Research Board Proceedings, Vol 13, 1933

Second, all capital expenditures for construction and/or purchase of fixed property, including, each year (a) highways, roads or streets, (b) buildings, (c) lands, (d) equipment, and (e) general administration and overheads applicable to the acquisition of the fixed property

Third, all expenditures for maintenance and operations, and depreciation and other operation costs, including, each year (a) direct maintenance expenditures, (b) general administration and overhead expenditures applicable to maintenance and operation, (c) depreciation on maintenance buildings and equipment, (d) depreciation on present fixed-capital roadway property

Fourth, all income from payments for road services, whether allocated by law directly (in full or in part), or indirectly by State appropriations, including, each year, (a) property highway taxes, (b) motor-vehicle license fees, (c) motor fuel taxes, (d) ton-mile taxes, (e) etc

Fifth, all income from other sources than payments for road services, including, each year (a) receipts from sales of bonds or other evidences of indebtedness, (b) receipts from appropriation of State funds other than payments for road services, (c) contributions from all sources including Federal aids and grants

Sixth, all income and expenditures for non-highway services rendered by the State highway department

Many public utility and other industrial enterprises have already developed standard uniform double entry accounting systems [5] In these, the double entry primary accounts are usually arranged in 6 main groups, as follows

1 Fixed-Capital Accounts
2 Operating Expense Accounts
3 Operating Revenue Accounts (an Operating Statement summarizes Groups 2 and 3)
4 Income Accounts (and an Income Statement)
5 Profit and Loss Accounts (and a Profit and Loss Statement)
6 Balance Sheet Accounts (and the Balance Sheet Statement)

The more the authors study the subject of uniform highway accountancy the more clearly they realize that the main fundamental requirements of a satisfactory system of uniform highway accounting are essentially the same as those for other utilities However, the accounting problems for the three groups, *highway income, operating revenue* and *profit and loss* are so different from those of the corresponding accounts for other utilities that it may (or in the end may not) be practicable to combine them in one group

Mainly to assist in their thinking and study, the authors have prepared a tentative lineup of possible double entry highway primary accounts, arranged in four main groups as shown in Appendix III,[6] which they now will proceed to discuss

ACCOUNT 1000 FIXED-CAPITAL INVESTMENT ACCOUNTS

The fixed-capital investment accounts are designed to show the investment of the department in all properties devoted to the maintenance, operation, and administration of highways The accounts should show the original cost and the true depreciation to date To the accounts in this group should be charged the cost of original highway construction, additions and betterments, and replacements, also values of right of way at time of acquisition, the cost of equipment, buildings, and other property pertaining thereto not consumed during the accounting year The accounts should be credited with depreciation

[5] Good examples are those prescribed by the Interstate Commerce Commission and by the Federal Power Commission for use by utilities subject to their regulatory authority

[6] The authors wish to emphasize the purely tentative character of Appendix III

Suitable property record ledgers will be required for each item or groups of similar items of property, on which the depreciations are credited as they accrue. For setting up these ledgers, the highways must be divided into unchangeable route sections, each with fixed geographical termini, these may be subdivided into subsections, of such length and characteristics that each may be accounted for as a unit.

Account 1100, Highways, Roads, and Streets. This account should include the actual original costs of all highways, including surfacing and base, earthwork, structures, devices, appurtenances, engineering, and right of way. For purposes of administration and reports, the account should first be subdivided into major accounts for each road system, such as primary roads, secondary roads, municipal streets, and other classifications that are appropriate within the State. Further, subdivision common to all of these major divisions by road systems should show the items of construction costs with particular reference to grouping by units according to depreciation rates and probably maintenance requirements, as indicated in Accounts 1111-1121 in Appendix III. The corresponding credit accounts for accrued depreciation are numbered 1111.1-1121.1.

Account 1200, Equipment. This account should include the cost of all equipment owned by the highway department with such subclassifications by type of equipment, function, and organization that are desirable from an administrative and cost control standpoint, as indicated in Accounts 1211-1215, 1221-1224, 1231-1233, 1241-1243 in Appendix III, with corresponding credit accounts for accrued depreciation.

Account 1300, Buildings. This account should include the cost of all buildings divided by location and general purpose, as indicated by Accounts 1310, 1320, 1330, 1340, Appendix III. Individual accounts should be kept for each building. All the building accounts should carry corresponding credit accounts for accrued depreciation.

Account 1400, Land Other than Right of Way. This account should include the cost of land which is owned, each parcel being kept by separate account, the parcels being grouped by use as indicated by Accounts 1410, 1420, 1430, 1440, 1450, Appendix III. Ordinarily land will be carried at purchase price without depreciation or appreciation adjustments. In the event, however, that the values change materially from the purchase price, the accounts should be adjusted. Quarries and mineral deposits should be written down each year in accordance with the depletion of the deposits. For these reasons, credit accounts for accrued depreciation or depletion are indicated for all the 1400 accounts in Appendix III.

Account 1500, General Administration and Overhead Investment. This account should include the cost of administration and such overhead expense that is chargeable to the construction of highways and acquisition of fixed property, as indicated by Accounts 1510, 1520, 1530, Appendix III. The expense of administration for a year should be prorated between fixed capital and operation expense. The portion charged to fixed capital should be depreciated annually the same as physical property, as indicated by credit Accounts 1510.1, 1520.1, 1530.1.

Account 1600, Suspense Accounts. This account should include the cost for properties which at the time may not be assigned to a final account. Particularly, it will be found convenient to charge to this account the monthly contractor's payments, pending settlement of the final estimates, and allocation of the final cost to highways, roads, and streets. See Accounts 1610, 1620, Appendix III.

ACCOUNT 2000 OPERATING EXPENSE ACCOUNTS

The accounts devised under the title of operating expense are those to show the expenses of furnishing the public with a highway transportation facility, including maintaining roadways, administration costs, and depreciation and maintenance of buildings and equipment The depreciation of fixed-capital highway, road and street property is also a highway operating expense (See accrued depreciation credit Accounts 1111 1-1121 1, and also Accounts 2411-2420, 2451-2453)

Account 2100, General Administration Expense This account should include the cost of administrating the department, including salaries of administrative officials, wages, office expenses, office building expense, office equipment expense, and other items of expense which are general in nature and difficult to charge to a specific construction or operation account at the time of payment A schedule of expense items to be included in this account will be developed after study of current practice and organization of the departments The total cost of general administration is ultimately chargeable in part to fixed capital and in part to operation expense For the total general administration cost, see Accounts 2100, Appendix III

Account 2200, Rental, Proration, and Suspense Accounts The group of accounts suggested under this major account are those expenses of service bureaus which are usually absorbed by other accounts and activities At the close of the accounting period each account would be closed out to operating expense or to fixed property accounts In most cases the expense would be closed out currently by use of purchase slip or service charges Each of the functions represented in the list of accounts is primarily an organization serving other functions of the department. See Accounts 2200, Appendix III

Account 2300, Highway Operation Expense This account should include the general operation expenses of the highway department, exclusive of those expenditures for fixed properties, and for other functions which are not directly in connection with the furnishing of a complete highway facility A separation of these expenditures is necessary in setting up a classification for use by all States, for the reason that the duties and responsibilities imposed by State laws are not common to all States See Accounts 2300, Appendix III.

Account 2400, Highway Depreciation This account should include the operation expense on account of depreciation of the highways and retirement of them The classifications will follow those included under the corresponding fixed capital accounts The account should also include the depreciation of general administration, research, and other general property accounts which are not classified with highways, equipment, buildings, and other tangible items. See Accounts 2411-2420 and 2451-2453, Appendix III

Account 2500, Other Highway Department Expense This account should include separate subaccounts for those functions of the highway department which are not directly related to the furnishing and maintenance of the highway system and which are not common to all States It will be important to show the expense of collection of income when such duty is that of the highway department See Accounts 2500, Appendix III

ACCOUNT 3000 OPERATING REVENUE AND INCOME ACCOUNTS

As already stated, the factual situation as to the operation revenues and other income receivable by the State highway departments differs in important particulars from the situation prevalent in the administration of other public utilities State highway departments do not receive payments of charges for road services direct from those to whom such services are rendered Collections for road services rendered are made by various State, county and Federal agencies, and go first into various public funds, they finally get to the highway departments by acts of State legislatures and of Congress Besides income from payments for road services rendered, highway department may receive large sums obtained by the sale of bonds Since these bonds and bond interest are usually to be paid from future receipts for future road services, it often becomes necessary to forecast such receipts for 15 to 20 years ahead Even merely for setting up construction programs and letting highway construction contracts, anticipated future highway revenues must be set up in some form of accounts against contractural obligations to show a clear picture of the true financial situation from time to time In addition, good highway management calls for setting up regular budget accounts in advance of anticipated revenues and anticipated expenditures

The authors are not including any *anticipated revenue* accounts [7] in Accounts 3000, in Appendix III, providing therein only for actual receipts These should be segregated into

1 Highway operating revenue
2 Income from sources other than highway operations

Income Accounts 3110, 3120, 3130, 3140, 3150, 3160, and 3170 include the receipts which constitute what might perhaps be fairly termed the highway operating revenue However, it is conceivable that legislative appropriation *might* be made of other State funds for highway construction as well as of funds received as payment for road services rendered

Credit Income Accounts 3181-3183, and Debit Income Accounts 3281-3282 together with Highway Operation Expense Accounts 2351-2356 (bond interest, discount and administrative debt expense), provide the necessary accounts for all receipts and expenditures on account of debt service

ACCOUNT 4000 GENERAL BALANCE SHEET ACCOUNTS

In addition to a "Highway Operating and Income Statement," such as is shown in Appendix IV, highway accountants from time to time will need to prepare "General Balance Sheet" statements, each giving a clear picture of the status of the highway system at some particular date The accounts needed for such balance sheets are listed in Appendix III, under Account 4000

These balance sheet accounts are listed in two columns, corresponding to the Asset and the Liability sides of the customary balance sheet form

Account 4100 includes the asset (debit) accounts

Accounts 4111-4116, show the current assets

Accounts 4121-4125 show the office, engineering, laboratory, stores and shop inventories assets

Accounts 4131-4133 show the contingent current assets Note that it is here that *estimated future incomes,* sometimes not receivable for one or more years, come into the highway accounts

Account 4140 shows sinking fund assets

Accounts 4151-4159 show the fixed-capital assets They may amount to several hundreds of millions of dollars in a single State Their main items are the actual highways (of which there may be many thousands of miles), the road equipment, the highway department shops and buildings of various kinds

[7] Provision is made, however, in Balance Sheet Accounts 4130 and 4220 for anticipated revenues and expenditures.

It is most customary to carry these at their actual costs *new* on the asset side of the balance sheet, with a corresponding liability account "Reserve for Depreciation" on the liability side The authors prefer the form shown in Appendix III, in which the physical assets are carried at their *present depreciated* values, instead of their *values new*, the *depreciation reserve account* disappearing from the liability side

Considering next the balance sheet liabilities items, carried under Account 4200

Accounts 4211-4216 show the current liabilities

Accounts 4221-4224 show the budgetary liabilities Like the contingent current asset accounts, these deal with *estimated future transactions* (in this case estimated future expenditures)

Accounts 4231, 4232 show the deferred liabilities

Accounts 4251, 4252 show the funded debts

CURRENT SURPLUS AND NET CAPITAL SURPLUS

The form of balance sheet shown in Appendix III, is a *dual balance sheet*

Subtracting the sum of current liabilities, budgetary liabilities and deferred liabilities from the sum of current assets, inventories and contingent current assets, we have a CURRENT SURPLUS

Subtracting the funded debt liability from the sum of sinking fund and fixed-capital assets we have a CAPITAL SURPLUS

HIGHWAY OPERATING AND INCOME STATEMENT

In public utility accountancy it is customary to provide for four "Periodic Statements," as follows

1 The operating statement
2 The income statement
3 The profit and loss statement
4 The balance sheet

In Appendix IV, the authors are presenting a form of combined "Highway Operating and Income Statement," which (or some improved equivalent) they are inclined to believe can be used to advantage in highway accountancy in place of the first three of the above customary four periodic statements With this probability in mind, the authors have not set up any highway "profit and loss" account in Appendix III

APPENDIX I

SUGGESTED CLASSIFICATION OF GENERAL LEDGER ACCOUNTS[1]

ASSETS

Current Assets
- Cash accounts—Separate accounts for each fund
- Accounts receivable—Miscellaneous items
- Reimbursements receivable
  - Federal aid—By apportionments
  - Counties, cities, etc
  - Railroads and other organizations
- Inventories
  - By appropriate subclassifications, such as materials and supplies, stores accounts, etc

Deferred Items
- Prepaid expense
- Suspense accounts

Fixed Assets
- Equipment
  - Office equipment
  - Shop equipment
  - Laboratory equipment
  - Engineering equipment
  - Highway construction and maintenance equipment
    - Major construction and maintenance equipment
    - Minor construction and maintenance equipment
  - Less Reserves for depreciation for each class of equipment

[1] From the report of the Committee on Uniform Accounting AASHO, Boston, Massachusetts, September 28, 1937

Land—Building sites, material deposits, quarries, pits, etc (Not including right of way)
Buildings
Less Reserve for depreciation on buildings
Work under construction
Highways and structures—Completed construction, additions and betterments, including right of way
Less Reserves for depreciation
(To be eventually arrived at by inventory valuation of the Highway System based on present value)
Experimental work, research and patents

LIABILITIES

Current Liabilities
Accounts payable
Short term notes or interest bearing warrants
Accrued interest and principal payments on long term bonds
Deferred Liabilities
Bonds or long term warrants—Principal outstanding
Advance deposits (By source)
Reserves for employees compensation insurance, retirement, etc

CAPITAL OUTLAY

Highways and structures
Equipment
Lands and buildings

WORKING CAPITAL FUNDS

Excess of current assets over current liabilities

INCOME

Highway users revenue (By sources, gas tax, license fees, etc)
Other revenue
Appropriations—General fund
Transfers from Other State Funds
Other sources
Federal government contributions and aid
By funds and source
All other contributions (By source, counties, cities, railroads, etc)
Miscellaneous income
Interest
Sales and services
Miscellaneous

EXPENDITURES

Administration
Central office (By organizational units, functions, and objective classifications)
Field office—same breakdown
Other than highway functions
Operating plants, shops, etc
Separate set of accounts for each organizational unit and function, such as warehouses, shops, testing laboratories, cement plants, equipment operations, stores
Maintenance of highways and structures
To be analyzed in subsidiary ledgers as outlined in the Maintenance section of the Manual
Construction of highways and structures (including acquisition of right of way)
To be analyzed in subsidiary ledgers as outlined in the Construction section of the Manual
Equipment purchases, construction and acquisition (By classes of equipment
Construction and acquisition of land and buildings, exclsive of right of way
Debt service—By subaccounts for each class of indebtedness or expense
Other departmental functions
Separate set of accounts by objective classification for each function such as highway patrol, revenue collection expense, State parks, toll bridges, toll ferries, etc

APPENDIX II

MAINTENANCE COST ACCOUNTING

From the Section on Maintenance Accounting as Proposed by the A A S H O Committee for the Manual on Uniform Highway Accounting

Definition of Maintenance Highway maintenance is the preserving and keeping of each type of roadway, roadside, structure and facility as nearly as possible in its original condition as constructed or as subsequently improved, and the operation of highway facilities and services to provide satisfactory and safe highway transportation

Classification of Accounts The account numbers and symbols which are used in this manual to identify the various classes of maintenance expenditures are *not recommended for* *adoption but are employed solely for convenience and by way of illustration*

Maintenance expenditures shall be accounted under the following main headings [1]

Account 100-A	Routine roadway surface operations—by surface types
Account 100-B	Special roadway surface operations—by surface types
Account 100-C	Shoulders and side approaches —by surface types
Account 100-D	Roadside and drainage
Account 100-E	Traffic services
Account 100-F	Snow, ice and sand control
Account 100-G	Structures
Account 100-H	Extraordinary repair and maintenance due to flood, storm, fire, earthquake, and major landslides
Account 100-I	Maintenance general expense

The total of the expenditures under these headings shall be accumulated in Account 100 Highway Maintenance

At the option of the State the above accounts may be further sub-divided by operations as indicated in the descriptions below It should be clearly understood that any subdivisions by operations in addition to the nine listed above are entirely optional with the State highway department

Account 100-A Routine Roadway Surface Operations To this account shall be charged all routine work performed on the roadway surface, base and subgrade, including

1 Patching holes, rough spots, ruts, blow-ups and raveled edges, sanding bleeding spots and spot sealing, providing temporary traffic ways during such operations
2 Dragging, blading, reshaping, scarifying, cleaning and raking, picking up oversize rock
3 Filling and trimming expansion joints and cracks

Account 100-B Special Roadway Surface Operations To this account shall be charged periodic work performed on the roadway surface, base and subgrade, usually by special crews, including

1 Application of dust palliatives when done annually or when the application does not result in a permanent improvement
2 Replacement of sand, sand-gravel, gravel, crushed stone, chat, etc , on the same or a similar type of surface
3 Reprocessing or reconditioning bituminous surfaces or shoulders when but little or no new oil or gravel material is added
4 Bituminous surface treating, seal coating, light road mixing operations, and major patching by special crews
5 Mud-jacking
6 Protection and handling of traffic during the above operations

When the surface type is changed by any of the above operations, or when the road is completely regraded, the operation shall be classed as construction

Reconditioning operations, additions of bituminous seal coats, surface treatments, mats or retreads which add in one operation ¾ inch or more to the thickness of the surface shall be classed as construction, the original surface being retired

The first seal coat shall be considered as construction if applied as within the following construction season, otherwise it will be classed as maintenance

Account 100-C Shoulders and Side Approaches To this account shall be charged expenditures for the repair and maintenance of shoulders and side approaches, including

1 Patching, dragging, blading, filling ruts and replacing washouts
2 Reseeding or resodding including original seeding on old work
3 Ribbon bituminous treatments (without excavation)
4 Second or subsequent bituminous treatment of shoulders and replacement of gravel or stone

Account 100-D Roadside and Drainage To this account shall be charged expenditures for the repair and maintenance of that portion of the right of way outside the limits of the traveled roadway surface and improved shoulders, including

1 The repairing of cuts, fills, slopes, washouts, and the removal of minor slides
2 Cleaning or retrenching drains, channels, culverts, and maintaining drainage structures of 20 feet in length or less
3 The removal and burning of weeds, removal of debris, planting or removal and trim-

[1] Note by the authors The general maintenance organization and plan for maintenance operations described in report (but not quoted herein) from which Appendix II is quoted contemplates taking into account depreciation of maintenance buildings and equipment by a rental plan (or some equivalent system)

In the opinion of the authors, depreciation on all other highway property (such as road surfaces, culverts, bridges, construction buildings and equipment, administration buildings and equipment) should likewise be debited to the proper accounts

ming of trees, brush and shrubs, and seeding and sodding to prevent erosion
4 Care and replacement of special roadside development projects
5 Repair and maintenance of sidewalks, dikes, retaining walls, riprap, pumping stations, slope pavements, right of way fences, and other similar structures and facilities

Account 100-E Traffic Services To this function shall be charged the repair and maintenance of those facilities wihch relate directly to the convenience and safety of the traveling public, including

1 Repairing, repainting and resetting of direction markers, route markers, signals, gates and other safety devices, and magnetic dragging to remove iron
2 Traffic lane and guide line painting
3 Repair and repainting of guard rails
4 Highway lighting and electricity for the operation of signals
5 Operation of comfort stations and picnic grounds
6 Detours not chargeable to construction or other maintenance operations

Account 100-F Snow, Ice and Sand Control To this account shall be charged all expenditures in connection with the removal of ice and drift material from the roadway, and preventive work in connection with such operations, including

1 Erection and removal of snow fences
2 Removal of snow and ice
3 Sanding ice surfaces
4 Snow and ice removal to open waterways
5 Removal of sand drifts

Account 100-G Structures To this account shall be charged expenditures incidental to the repair and maintenance and operation of bridges, subways, tunnels, overhead grade separations and other structures having a length of more than 20 feet

Each such structure may at the time of construction be assigned a number, which number will be used in reporting expenditures on any of these structures The subsidiary ledger under the account "Structures" will consist of an account for each structure and from this detail summaries of expenditures for each class of structure can be compiled if desired

Charges to this account shall include operating expenses of bridges, drawbridges and ferries

Account 100-H Extraordinary Maintenance To this account shall be charged special repairs and maintenance of the roadway surface, roadbed, shoulders, roadside, drainage facilities, safety devices or structures due to flood, storm, fire, earthquake, major landslide, or other catastrophe Expenses in connection with the handling and protection of traffic during such emergency shall also be charged to this function

Account 100-I Maintenance General Expense To this account shall be charged maintenance expenditures of a general nature which cannot at the time be charged direct to roadway sections, including

1 Prorata of district office and/or headquarters office expense chargeable to maintenance
2 Field maintenance supervision
3 Purchase and repair of small tools, rental charges on inactive equipment, and similar items The account will serve as a suspense account during the accounting period, at the end of which the entire balance will be prorated to roadway sections in the manner provided in a later section of the manual

For administrative purposes it will probably be found desirable to have an analysis of the charges going into this account, and it is therefore suggested that the following information be available from the records

DO District and headquarters office changes
MS Field maintenance supervision
ST Small tools
RT Repair of small tools
SE Sundry expense

APPENDIX III

A SUGGESTED LIST OF PRIMARY ACCOUNTS FOR HIGHWAY DEPARTMENTS

1000 FIXED CAPITAL INVESTMENT ACCOUNTS

Each capital investment debit account should carry an accrued depreciation credit account These are shown below For example, Account 1111 1, Surfacing and base—Depreciation

1100 Highways, Roads and Streets

1110 Rural mileage, State primary system
  1111 Surfacing and base
    1111 1 Surfacing and base—reserve for depreciation

1112 Improved shoulders
1112.1 Improved shoulders—reserve for depreciation
1113 Earthwork
1113.1 Earthwork—reserve for depreciation
1114 Earthwork protective structures
1114.1 Earthwork protective structures—reserve for depreciation
1115 Culverts and drainage structures
1115.1 Culverts and drainage structures—reserve for depreciation
1116 Traffic services
1116.1 Traffic services—reserve for depreciation
1117 Bridges and tunnels
1117.1 Bridges and tunnels—re- for depreciation
1118 Grade separation structures
1118.1 Grade separation structures—reserve for depreciation
1119 Engineering
1119.1 Engineering—reserve for depreciation
1120 Right-of-way land purchased
1120.1 Right-of-way land purchased—reserve for depreciation
1121 Right-of-way land-easements
1121.1 Right-of-way land-easements—reserve for depreciation

1200 Equipment

1210 Road equipment
1211 Roadway motor equipment
1211.1 Roadway motor equipment—reserve for depreciation
1212 Roadway non-motor equipment
1212.1 Roadway non-motor equipment—reserve for depreciation
1213 Motor trucks
1213.1 Motor trucks-reserve for depreciation
1214 Automobiles
1214.1 Automobiles—reserve for depreciation
1215 Road tools
1215.1 Road tools—reserve for depreciation
1220 Shop and service equipment
1221 Shop equipment
1221.1 Shop equipment—reserve for depreciation
1222 Shop machines
1222.1 Shop machines—reserve for depreciation
1223 Shop tools
1223.1 Shop tools—reserve for depreciation
1224 Warehouse equipment
1224.1 Warehouse equipment—reserve for depreciation
1230 Office and administrative equipment
1231 Office furniture, files and equipment
1231.1 Office furniture, files, and equipment—reserve for depreciation
1232 Typewriters, calculators, and office machines
1232.1 Typewriters, calculators, and office machines—reserve for depreciation
1233 Communication systems
1233.1 Communication systems—reserve for depreciation
1240 Engineering and laboratory equipment
1241 Field engineering instruments and tools
1241.1 Field engineering instruments and tools—reserve for depreciation
1242 Office engineering instruments
1242.1 Office engineerings instruments—reserve for depreciation
1243 Laboratory equipment and apparatus
1243.1 Laboratory equipment and apparatus—reserve for depreciation

1300 Buildings

1310 Central office buildings
1310.1 Central office buildings—reserve for depreciation
1320 Central shop and warehouse buildings
1320.1 Central shop and warehouse buildings—reserve for depreciation
1330 Field office buildings
1330.1 Field office buildings—reserve for depreciation
1340 Field shop and warehouse buildings
1340.1 Field shop and warehouse buildings—reserve for depreciation

1400 Land Other than R/W

1401 Central office building land
1410.1 Central office building land—reserve for depreciation

1420 Field building land
- 1420 1 Field building—reserve for depreciation

1430 Other land
- 1430 1 Other land—reserve for depreciation

1440 Gravel deposits
- 1440 1 Gravel deposits—reserve for depreciation

1450 Rock quarries
- 1450 1 Rock quarries—reserve for depreciation

1500 General Administration and Overhead Investment

1510 General administration investment
- 1510 1 General administration—reserve for depreciation

1520 Research and planning
- 1520 1 Research and planning—reserve for depreciation

1530 General engineering not prorated
- 1530 1 Undistributed engineering—reserve for depreciation

1600 Suspense Accounts

1610 Earnings on work under construction (suspense until final costs are spread to the proper asset account)

1620 Loss of construction due to catastrophe (suspense during reconstruction)

2000 OPERATING EXPENSE ACCOUNTS

2100 General Administration Expense

2110 Commission and State expense
- 2111 Salaries
- 2112 Traveling expense
- 2113 Other

2120 General office expense
- 2121 Salaries and wages
- 2122 Traveling expense
- 2123 Communication
- 2124 Supplies and expense
- 2125 Printing and binding
- 2126 Repair of office furniture, files, and equipment
- 2127 Depreciation of office furniture, files, and equipment
- 2128 Repair and rental of office machines
- 2129 Depreciation of office machines

2130 General accounting expense
- 2131 Salaries and wages
- 2132 Traveling expense
- 2133 Communication
- 2134 Supplies and expense
- 2135 Printing and binding
- 2136 Repair of office furniture and equipment
- 2137 Depreciation of office furniture and equipment
- 2138 Repair and rental of office machines
- 2139 Depreciation of office machines

2140 Legal expense
- 2141 Salaries and services
- 2142 Traveling expense
- 2143 Supplies and expense

2150 Office building expense
- 2151 Salaries and wages
- 2152 Operation supplies
- 2153 Utility services
- 2154 Repair and maintenance
- 2155 Building depreciation
- 2156 Heating or heating plant expense

2160 Other administrative expense
- 2161 Injuries and damages
- 2162 Other

2200 Rental, Proration and Suspense Accounts

(This group of service accounts will be ultimately closed out to other accounts)

2210 Stores and stocks
- 2211 Salaries and wages
- 2212 Traveling expenses
- 2213 Supplies and expense
- 2214 Utility services
- 2215 Building operation
- 2216 Building repairs
- 2217 Building depreciation
- 2218 Equipment repairs
- 2219 Equipment depreciation

2220 Material deposits
- 2221 Salaries and wages
- 2222 Traveling expense
- 2223 Equipment operation and repair
- 2224 Equipment depreciation
- 2225 Depletion of deposits

2230 Blue printing and duplicating
- 2231 Salaries and wages
- 2232 Supplies and expense
- 2233 Equipment repair
- 2234 Equipment depreciation

2240 Shop overhead expense (to be prorated to operation of each equipment unit)
- 2241 Salaries and wages
- 2242 Office supplies and expense
- 2243 Traveling expense
- 2244 Machine and equipment repairs
- 2245 Machine and equipment depreciation
- 2246 Tools and supplies
- 2247 Building operation and utilities
- 2248 Building repairs and maintenance
- 2249 Building depreciation

2250. Equipment operating expense (by equipment operating unit)
- 2251. Wages, repair work
- 2252. Parts, repair
- 2253. Grease and service
- 2254. Fuel
- 2255. Oil
- 2256. Tires
- 2257. Depreciation
- 2258. Shop overhead

2260. Engineering expenses (field and office)
- 2261. Salaries and wages
- 2262. Traveling expense
- 2263. Office supplies and expense
- 2264. Tools and engineering supplies
- 2265. Office files and equipment repair
- 2266. Office files and equipment depreciation
- 2267. Instrument and machine repairs
- 2268. Instrument and machine depreciation

2270. Testing laboratory
- 2271. Salaries and wages
- 2272. Traveling expense
- 2273. Office supplies and expense
- 2274. Laboratory tools and supplies
- 2275. Instrument and equipment repairs
- 2276. Instrument and equipment depreciation
- 2277. Building operation and utilities
- 2278. Building repairs and maintenance
- 2279. Building depreciation

2280. Research and planning
- 2281. Salaries and wages
- 2282. Traveling expense
- 2283. Office supplies and expense
- 2284. Research tools and supplies
- 2285. Equipment repairs
- 2286. Equipment depreciation
- 2287. Patent expense

2290. Radio operation
- 2291. Salaries and wages
- 2292. Traveling expense
- 2293. Supplies and expense
- 2294. Repair and maintenance of equipment
- 2295. Depreciation of equipment

2300. Highway Operation Expense

2310. General administration proration

2320. Highway maintenance (show labor, materials, equipment)
- 2321. Routine roadway surface operation
- 2322. Special roadway surface operations
- 2323. Shoulders and side approaches
- 2324. Roadside and drainage
- 2325. Traffic services
- 2326. Snow, ice and sand control
- 2327. Bridges, viaducts and tunnels
- 2328. Extraordinary repair and maintenance due to catastrophe
- 2329. Maintenance general expense

2330. Operation of drawbridge, toll and ferry facilities (by each facility) (structure maintenance under highway maintenance account 2320)
- 2331. Salaries and wages
- 2332. Traveling expense
- 2333. Supplies and expense
- 2334. Building operation and repair
- 2335. Building depreciation

2340. Tourist bureau expense
- 2341. Salaries and wages
- 2342. Traveling expense
- 2343. Supplies and expense
- 2344. Printing and advertising

2350. Debt service general expense
- 2351. Salaries and wages
- 2352. Office expense and printing
- 2353. Legal expense
- 2354. Bond interest
- 2355. Bond discount
- 2356. Short term interest

2400. Highway Depreciation

2410. Depreciation of highway investment
- 2411. Surfacing and base
- 2412. Improved shoulders
- 2413. Earthwork
- 2414. Earthwork protective structures
- 2415. Culverts and draingage structures
- 2416. Traffic services
- 2417. Bridges and tunnels
- 2418. Grade separation structures
- 2419. Engineering
- 2420. Right-of-way

2430. Transfer of highways to other jurisdictions
- 2431. Surfacing and base
- 2432. Improved shoulders
- 2433. Earthwork
- 2434. Earthwork protective structures
- 2435. Culverts and drainage structures
- 2436. Traffic services
- 2437. Bridges and tunnels
- 2438. Grade separation structures
- 2439. Engineering
- 2440. Right-of-way

2450. Other depreciation expense
- 2451. Depreciation of general administration
- 2452. Depreciation of research and planning
- 2453. Depreciation of undistributed engineering

2500. Other Highway Department Expense

2510. Revenue collection expense (for each revenue or income source)
- 2511. Salaries and wages
- 2512. Traveling expense
- 2513. Communication
- 2514. Office supplies and expense
- 2515. Printing and binding
- 2516. Repair of office furniture, files and equipment
- 2517. Depreciation of office furniture, files and equipment
- 2518. Repair and rental of office machine
- 2519. Depreciation of office machine
- 2520. Refunds due to errors
- 2521. Exemption refunds

2530. Highway patrol
- 2531. Salaries and wages
- 2532. Traveling expense
- 2533. Office supplies and expense
- 2534. Automobile operation (patrol)
- 2535. Uniforms
- 2536. Tools and supplies
- 2537. Legal expense
- 2538. Building operation
- 2539. Building repairs and maintenance
- 2540. Building depreciation
- 2541. Training school and personnel

2550. Other expense
- 2551. Testing and engineering expense for others
- 2552. County road supervision
- 2553. Maintenance of State institution roads
- 2554. Outdoor advertising regulation
- 2555. Operation of State parks and park roads

3000. OPERATING REVENUE AND INCOME ACCOUNTS

3100. Credits

3110. Highway user—general
- 3111. Vehicle license fee
- 3112. Motor fuel tax
- 3113. Driver license

3120. Highway user—specific
- 3121. Ton-mile tax
- 3122. Port of entry
- 3123. Bridge and ferry tolls
- 3124. Special permits
- 3125. Fines and penalties

3130. U. S. Government aid
- 3131. Federal aid
- 3132. Flood relief
- 3133. Emergency

3140. Other government aid and joint operations
- 3141. Other States
- 3142. Counties
- 3143. Towns and cities

3150. Legislative appropriations from general funds and property taxation
- 3151. Legislative appropriations
- 3152. Special assessments
- 3153. Benefit districts
- 3154. Property road taxes

3160. Miscellaneous income
- 3161. Donations and grants
- 3162. Railways
- 3163. Oil and land royalties
- 3164. Interest on deposits
- 3165. Sales, service and incidentals
- 3166. Rentals (buildings and equipment)
- 3167. Other income

3170. Recepits other than cash
- 3171. Receipts other than cash

3180. Income from borrowing
- 3181. Bond sales
- 3182. Bond premiums
- 3183. Short term loans

3200. Debits

1000. Fixed Capital Investment expenditures
- 1100. Highways, roads, and streets
- 1200. Equipment
- 1300. Buildings
- 1400. Land other than R/W

2000. Operating expense accounts
- 2100. General administration expense
- 2300. Highway operation expense
- 2400. Highway depreciation expense
- 2500. Other highway department expense

3280. Extinguishment of debt principal
- 3281. Bond payments (principal)
- 3282. Short time loan payments (principal)

4000. GENERAL BALANCE SHEET ACCOUNTS

4200. Liabilities and other Credits

4210. Current liabilities
- 4211. General accounts payable
- 4212. Wages payable
- 4213. Current notes payable
- 4214. Short term debts
- 4215. Unearned construction and maintenance contracts
- 4216. Interest due on funded debt

4220. Budgetary liabilities
- 4221. Administrative budget balance

4222. Construction budget balance not obligated by contracts or work orders
4223. Maintenance budget balance
4224. Other budget balance not obligated

4230. Deferred liabilities
4231. Advance deposits
4232. Reserve for employees insurance and retirement

4240. CURRENT SURPLUS

4250. Funded debt
4251. Highway bonds
4252. Long term warrants

4260. NET CAPITAL SURPLUS

4100. Assets and other Debits

4110. Current assets
4111. Cash (by funds)
4112. General accounts receivable
4113. Federal and other reimbursements receivable (by accounts)
4114. Interest receivable
4115. Notes receivable
4116. Prepaid expense

4120. Inventories
4121. Office supplies
4122. Engineering tools and supplies
4123. Laboratory supplies
4124. Stores and stocks
4125. Shop parts and supplies

4130. Contingent current assets
4131. Appropriations not received
4132. Anticipated income
4133. Federal allotments not received

4140. Sinking funds

4150. Fixed capital assets
4151. Office and administrative equipment
Less reserve for depreciation
4152. Engineering and laboratory equipment
Less reserve for depreciation
4153. Shop and service equipment
Less reserve for depreciation
4154. Road equipment
Less reserve for depreciation
4155. Land other than R/W
Less reserve for depreciation
4156. Buildings
Less reserve for depreciation
4157. General administration and overheads
Less reserve for depreciation
4158. Highways, roads, and streets
Less reserve for depreciation
4159. Earnings on work under construction

APPENDIX IV

FORM OF HIGHWAY INCOME AND OPERATING STATEMENT

The following suggested classified form of income and operating statement is intended to show a classified summary of the significant financial results of the operations (including construction) of a State highway system during a given accounting period (usually one year).

I. HIGHWAY OPERATING RECEIPTS AND REVENUE

	Cash received	Appropriation income, or accounts due	Total earned revenue
3111. Vehicle license fee	......	......	......
3112. Motor-fuel tax	......	......	......
3113. Driver license	......	......	......
3121. Ton-mile tax	......	......	......
3122. Port of entry	......	......	......
3123. Bridge and ferry tolls	......	......	......
3124. Special permits	......	......	......
3125. Fines and penalties	......	......	......
3131. Federal aid	......	......	......
3132. Federal flood relief	......	......	......
3133. Federal emergency	......	......	......
3140. Other government aid	......	......	......
3151. Legislative appropriation from general fund	......	......	......
3152. Special assessments	......	......	......
3153. Benefit districts	......	......	......
3154. Property road tax	......	......	......
3160. Miscellaneous income	......	......	......
3170. Revenue, other than cash	......	......	......
Total revenue	......	......	......

II. HIGHWAY OPERATING EXPENDITURES
2310. General administration proration [1] ....
2320. Highway maintenance [1] ....
2330. Operation of toll facilities [1] ....
2340. Tourist Bureau expense ....
2500. Other highway department expense [1] ....

Total highway operating expenditures ....

III. HIGHWAY OPERATING RETURN [2]

IV. HIGHWAY DEPRECIATION
1100.1. Depreciation of highways ....
1200.1. Depreciation of equipment ....
1300.1. Depreciation of buildings ....
1400.1. Depreciation of land, gravel and rock deposits ....
1500.1. Depreciation of general administration and overheads ....

Total depreciation expense appropriation for depreciation reserve highway construction, and purchase of equipment and buildings.. ......

V. HIGHWAY NET OPERATING RETURN [3]
Operating return ....
Less interest cost charge [4] ....
Highway operating income compared with cost, surplus or deficit ....

VI. DISPOSITION OF HIGHWAY NET OPERATING RETURN
1. Debt service payments and expense
2351–2353. Debt service general expense ....
2354. Bond interest ....
2356. Short term interest ....
3281. Bond payments (principal) ....
3282. Short time loans payments (principal) ....

Total debt service payments ....

2. Fixed-Capital Investment purchases [5]
1100. Investment in highway, road, and street construction ....
1200. Investments in equipment ....
1300. Investments in buildings ....
1400. Investments in land (other than R/W), gravel deposits, rock quarries ....
1500. Investment in general administration and overheads [6] ....
1600. Earnings on work under construction ....

Total fixed-capital investment during year ....
Total expenditures of highway net operating return ....
Balance ....

VII. CONSTRUCTION WITH BORROWED FUNDS
1. Receipts
Balance January 1 (or other beginning date) ....
3181, 3182. Bonds and bond premiums (by issues) ....
3182. Short term loans (by loans) ....

Total receipts ....

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[1] Excluding depreciation.
[2] Highway operating return = operating income — operating expenditures.
[3] Highway net operating return = operation return — highway depreciation accruing during the accounting period.
[4] Interest cost charge = interest on fixed capital assets and working capital.
[5] Less investments from borrowed funds and from depreciation expense appropriation.
[6] Proration of general administration expense (Account 2100) and of suspense accounts (Account 2200).

2. Investments paid for by borrowed funds	
1100. Highways, roads, and streets	......
1200. Equipment	......
1300. Buildings	......
1400. Land, other than R/W	......
1500. Administration and overhead	......
1600. Earnings on work under construction	......
Total investments	......
Balance December 31 (or other closing date)	......
VIII. TOTAL FIXED-CAPITAL INVESTMENTS DURING YEAR	
1100. Highway construction paid for from borrowed funds	......
1100. Depreciation reserve highway construction	......
1100. Net operation return highway construction	......
Total investments in highway construction	......
1200. Investments in equipment	......
1300. Investments in buildings	......
14.00 Investments in land (other than R/W), gravel deposits, rock quarries	......
1500. Administration and overheads	......
1600. Earnings on work under construction	......
Total investment other than highways	......
Total Fixed-Capital Investments During Year	......

RURAL MAIL CARRIER MOTOR VEHICLE OPERATING COSTS ON VARIOUS TYPES OF ROAD SURFACES

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SYNOPSIS

Operating cost records for 160 cars operated by rural mail carriers in Iowa and in Indiana have been assembled, summarized, and analyzed for the purpose of determining the operating cost differentials for various types of road surfaces The data show that the average unit operating cost differentials are from 5 to 10 times greater than the average unit motor vehicle tax contribution for the construction and maintenance of roads, and therefore should be considered as the most important factor in selecting the type of surface to be used for given traffic conditions

From the detailed daily record of these cars covering every phase of operation such as miles of travel on each surface, rate of travel, weather, number of stops, load, gasoline and oil consumption, tire expense, maintenance costs, garage rental, license fees, taxes, insurance, depreciation, interest, and extra help The unit operating costs were determined for each type of surface

From the plotted data the average cost of operation for the year-round condition was found to be about 8 cents per mile for earth or natural soil roads, 5 cents per mile for untreated gravel surface, and 3½ cents per mile for portland cement concrete or bituminous macadam surfaces The operating cost was 2¼ cents per mile less in summer than in winter for earth roads, 1¾ cents per mile for untreated gravel, and 1 cent per mile for pavement The average annual travel for all cars was 10,919 miles, for cars operating entirely on earth roads, it was 1,500 miles, for cars operating on gravel roads, 11,000 miles per year, and on paved roads it was 14,000 miles per year The average miles of route covered per hour, including stops, was about 14 on gravel and pavement and 8 per hour on earth roads for the year-round condition During the summer it was 16 miles per hour on paving and gravel as compared to 10 miles per hour on earth roads In the winter it was 12 miles per hour on paving and gravel as compared to 6½ miles per hour on earth roads If it is assumed that the value of the mail carriers' time is 50 cents per hour, the paved or gravel road would then provide a time saving 2⅔ cents per mile over the earth road for the year round condition

In the statistical analysis, the average cost of gasoline, oil, tires and maintenance for the year-round condition was 3.17 cents per mile for earth roads, 2.46 cents per mile for gravel roads, and 1.73 cents per mile for paving The cost of gasoline, oil and maintenance for the winter months was 3.44 cents per mile for earth, 2.56 cents per mile for gravel, and 2.06 cents per mile for paving For the summer months these costs were 1.97 cents per mile for earth, 1.71 cents per mile for gravel and 1.49 cents per mile for paving

Applying these data to determine the traffic volume necessary to justify changing from an unimproved earth road to an improved gravel surfaced road for a surfaced road for a typical Iowa county, it was found that 20 vehicles a day are required if operating costs only are considered and 3 vehicles a day if a time value of 2⅔ cents per mile is added

Cost records for 160 cars operated by rural mail carriers in Iowa and in Indiana have been assembled, summarized, and analyzed for the purpose of determining the operating cost differentials for various types of road surfaces This study, which is being conducted by the Iowa Engineering Experiment Station, will serve as a pilot study for a more comprehensive program now being planned as a cooperative project with the Bureau of Public Roads to determine as nearly as possible the true operating cost differentials for various types of road surfaces

Although the Iowa Engineering Experiment Station has conducted studies along these lines over a period of more than fifteen years under the direction of Dean T. R. Agg, these studies have provided only partial information in regard to operation costs, generally covering only such items as tractive resistance, fuel consumption, and tire wear, for various surfaces, or covering all items related to operating costs for cars and trucks but without reference to surface types. In many cases road tests were conducted only under favorable weather conditions and over short test courses. Or in the case of projects where studies were made of cost records turned over to the Station by car or truck owners, practically no information was available to indicate the mileage traveled on various types of surfaces. Accordingly, this study is the first comprehensive car cost study which has come to the writer's attention in which the mileage traveled on each type of surface is known and in which the cars operated practically every day of the year in all kinds of weather.

While it is true that the operation of a car on a rural mail route is a highly specialized operation and should not be considered as typical of the operation of cars on all rural highways, certainly not on main state highways, it seems fair to state that this type of operation is quite common on a large mileage of local county roads, land service roads, or farm-to-market roads which are in great need of improvement. Rural mail service, school bus service, the collection and delivery of farm products such as milk, eggs, butter, poultry, meat, etc., and the country nurse or doctor service, all have much in common as far as the nature of their motor vehicle operation is concerned and furthermore, they constitute a large portion of the traffic on these roads. Stops in all of these services are frequent, and speeds are generally low, rarely exceeding 30 or 40 miles per hour. The nature of the service in practically all cases requires operation every day of the year in all kinds of weather. Therefore, a study of operating costs for cars operated by mail carriers may be considered as fairly representative of operation on the purely local or farm-to-market roads. That such a study is important becomes apparent when one realizes that approximately two million of the three million miles of road in this country may be included in this class.

During the past 20 years of intensive road building the highway engineer has been concerned mainly with the improvement of the important state and county routes which carry the large bulk of traffic. Now that the main highways are practically all improved with all-weather surfacing, considerable attention is being given to extending the surfacing program to farm-to-market roads. In certain eastern states, notably in Pennsylvania, this work has gone forward at a rapid pace. However, in many of the midwest, far west, and southern states, much remains to be done. Funds for road improvement in these more sparsely populated areas are not as easily obtained as in the eastern states. The value of road improvement and improved road surface construction is still not fully realized in these states. Renewed emphasis might profitably be given to that well known roadbuilder's slogan "car owners pay for good roads whether they have them or not." This implies that the car costs on unimproved roads are considerably higher than on improved roads due to additional fuel required, extra tire wear, maintenance expense and other costs which in many cases far exceed the cost to the car owner in the form of extra taxes for improving the road.

In the case of main highways where the traffic volume was large, even 10 or 15 years ago, it was easy to show that the savings in operating cost justified the improvement. Refined and carefully controlled studies of operating costs were not necessary. Today the highway engineer is called on to make decisions in regard to the selection of surfacing improvements for secondary roads on which the traffic is much lighter and for which a more careful analysis of all the cost factors is necessary than on the main state highways, if a wise economic selection of the type of surface improvement is to be made.

The differences in operation cost on stabilized surfaces, such as soil, cement, or oil stabilized roads, and on the higher types of surfaces, such as the various bituminous surfaces, portland cement concrete, brick, etc., are small and are difficult to measure. No attempt was made in this study to measure these differences. The cars in this study operated largely on portland cement concrete, untreated gravel and on unimproved earth or natural soil surfaces. The differences in operating costs on these surfaces were very marked. In fact, the differences in the operation costs for the year round conditon on earth as compared to gravel or paved surfacing far exceed previous estimates. If the low annual mileage of cars traveling on earth roads is taken into account and if the time factor is evaluated those particular data indicate that a traffic volume as low as three vehicles per day justifies an investment of $1,000 per mile for road surfacing improvements for road conditions similar to those in Iowa on its local farm-to-market roads.

"THE MAILS MUST GO THROUGH!"

Road improvements on our main highways are so general that many road users and, indeed, many highway engineers too, have forgotten what mud roads are like. They are not aware of the many inconveniences, of the extra equipment required, and extra costs incurred on mud roads such as are shown in Figures 1 to 4. Uncle Sam says, "The mails must go through!" and the rural mail carrier must take the roads as he finds them. The reports from a large number of carriers showed that special equipment was being used by these carriers to operate on mud roads and snow-covered roads which are practically impassable if standard equipment is used.

In Figure 1 a special mud car is shown using large diameter rear wheels to permit operating in the deep ruts formed on mud roads. The extra clearance between the wheels and the fenders prevents the mud from accumulating under the fender to the point where it will interfere with the free action of the wheel. This open type fender makes it easy to clean out the mud on a gumbo road where the mud has a tendency to stick to the tires and roll up in thick layers which must be removed with a spade at frequent intervals. The mud condition and deep ruts shown in Figure 2 are quite common on unimproved local roads throughout the middlewest today. One ingenious mail carrier recognized that there should be a close relation between mud roads and corrugated metal pipe. He decided that if the county engineer failed to use it effectively, then he would use it to prevent being bogged down in the mud by clamping about a 12-inch section of the pipe to each rear wheel forming an extension to the wheel which would provide the necessary support after the tire pene-

Figure 1. Special "Mud" Car Used in Jefferson County

trated the mud to the depth of the corrugated metal pipe as shown in Figure 3. Several mail carriers reported the use of the special Snowmobile equipment shown in Figure 4. The Snowmobile equipment is effective both on snow-covered and muddy roads. But it is quite

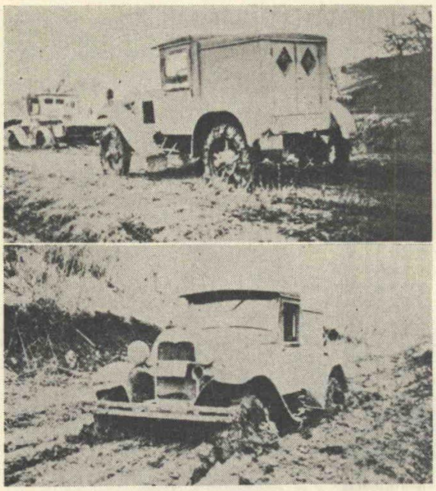

Figure 2. High Clearance Axles in Use on Jackson County Roads

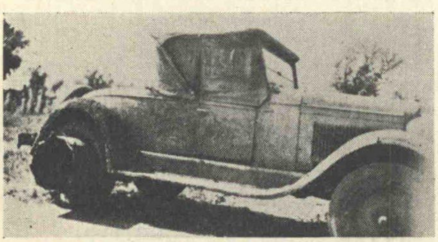

Figure 3. Corrugated Metal Culvert Pipe Used as Wheel Extension for Deep Mud on Buchanan County Roads.

evident that the operating costs for equipment of this type or under these conditions are appreciably higher than on improved roads with all-weather surfaces. Also the average speeds are very much lower for this equipment and on these roads than for standard car equipment on hard road surfaces. How much greater these costs and the accompanying delays are on earth roads than on all-weather surfaces is a question which has not been fully answered up to this time. The analysis of complete daily records of operating costs extending over a full year, as were obtained in the mail carrier reports, provides some valuable factual evidence in regard to those costs and delays.

CAR COST STUDY REPORT FORMS

Route and Car Descriptions

The first step was to send out Form 1 which was filled out by the carrier who agreed to furnish us with a daily record of all car operation items and costs both on and off the route for at least 12

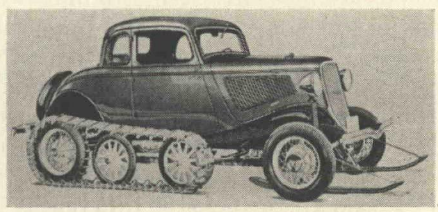

Figure 4. Snowmobile with Ski Runners in Front and Traction Belt Drive on Rear, Used to Combat Mud and Snow.

months. This Form provides a complete description of the route and of the car.

Separate forms were made for each car used or for a new car if a change was made during the year.

Daily Record Book

A daily record book was sent out to each carrier. This book provided a daily record of the type shown in Form 2 for one full month. On the front cover of each book the month and the year were filled out by the owner, together with his name and address, the make, year model, number of cylinders and the license number of the car. A new book was started the first day of each month and the book for the previous month was sent in to the Experiment Station at Ames.

It should be noted that the daily record was fairly easy to keep yet it provided all the necessary information to conduct the study Thus for each day the speedometer reading was given, followed by the temperature and weather conditions which were checked, the pounds of mail carried, the travel time including stops, the miles driven on route, the itemized expenditures such as gasoline, oil, tires, repairs, chains, greasing and other items of expense The hours of personal labor on the car each day were listed Remarks covering such items as road condition, delays, detours, accidents, extra help, chains used, etc, were also noted At the bottom of the page the miles of road driven off the route were classified according to the four road surface types previously defined

On the inside of the back cover Form 3 was provided to keep a record of tire mileage, the make, cost and a record of tire changes during the month

Monthly Summary Sheets

After the daily record book was sent in, a preliminary monthly summary was made and entered upon Form 4 On this form a careful record of the route and off route mileage was entered and classified as to surface type for the total mileage operated each month A monthly summary of all operation items was listed on Form 5 This provides for a summary of route and off route mileage, the amount of cost of gasoline used, oil, tire cost, maintenance costs which included tire chains, anti freeze, washing, greasing, repairs, and personal labor at 40 cents per hour, the cost of extra help, the miles of travel per hour (including stops) and miscellaneous items which were listed under remarks

Final Quarterly and Annual Summary Sheet

After a full year's record was completed, a final quarterly and yearly summary was made for each car and entered upon Form 6 The quarterly summary covered the same items listed on the monthly summary *Extra Help* A distinction was made between extra help with and without the car Extra help with the car may generally be interpreted to mean that the roads were passable but travel was so slow and so difficult that the carrier needed assistance to cover the route in a reasonable length of time Extra help without the car may be interpreted to mean that the roads were impassable either because of snow or mud and that the car had to be replaced with some other form of transportation as the use of a horse or a team and wagon, or of several men on foot from points on the route which were accessible

Recorded Costs In the summary for the year both recorded and adjusted costs are tabulated The recorded costs are merely the sum of the costs set down in the daily record books Since the record was kept for only one year, the recorded cost did not provide a complete and accurate record from which a true unit cost of operation could be computed In many cases no records were entered covering such items as tire replacement, garage rental, insurance, depreciation, and interest or if such items were entered they were not representative of the actual cost for the 12 months during which this study was made In many cases considerable tire data were given which were helpful in determining the average tire life for operation on various surface types, but this information rarely, if ever, was given in such a way that the recorded cost indicated accurately the cost for the year for the definite mileage over which the car was operated The records also provide information in regard to depreciation since the trade-in value for a number of cars was given Unfortunately the depreciation based on these trade-in values generally covered periods greater than that over which the cost records were kept In order to arrive at a uniform basis for comparing unit operating costs, adjusted costs were computed for such items as tires, garage, insurance, depreciation and interest The explanation of how these adjusted costs were computed is given below

Adjusted Costs The adjusted costs for gasoline, oil, and maintenance were the same as the recorded costs This was also true for the license fee except where penalties were paid, no penalties being included in the adjusted cost

Tire costs were computed on the basis of the miles of travel on the types of surface over which the tires were used The percentage of total miles traveled on hard surfaced roads was calculated and the average life of the tires taken from Table 1 The average tire life shown in this table was determined from a study of the tire mileage data reported on Form 3 and from tire wear tests conducted by the Station as reported at the last annual meeting [1]

The tire cost in cents per mile was computed on the basis of the average mileage per tire given in the table and an average tire replacement cost of $15 per tire To obtain the adjusted tire cost it was only necessary to select the correct value for unit cost in Table 1, multiply it by the total miles driven, and add to this the tire repairs as recorded in the Final Monthly Summary on Form 5

Many carriers made no charge for garage rental To arrive at a uniform adjusted cost, the rental values which were recorded were averaged It was found that $2 50 per month was close to the average and this figure was selected as a fair rental charge for rural mail carrier cars

[1] "Gasoline, tire wear, and coefficients of friction on various road surfaces" by R A Moyer and H W Tillapaugh Proc Highway Research Board, Vol 16 (1936)

Insurance charges were figured on the basis of rates at Ames, Iowa, during 1936 The coverage included $5,000 property damage, and $10,000 and $20,000 public liability The standard rates for fire, theft, and tornado were applied for cars less than 7 years old For cars older than 7 years, the depreciated value of the cars is so low that the cost of insurance for fire, theft, and tornado is not justified

To arrive at a fair method of determining depreciation a study was made of used car values for various standard makes of cars As a result of this study, Table 2 was made up showing depreciated values of Iowa and Indiana cars for various ages from a new car to a car 7½ years old It should be noted in this table that the depreciated value is given in percent of the value new for each half year up to 7½ years Beyond this age the depreciated value was assumed to be equal to the salvage value and, therefore, the depreciation was zero for all such cases When no delivered price was given, it was obtained by multiplying the list price by 1 24 for Iowa and 1 22 for Indiana To compute the depreciation over any period, it was necessary to know the age of the car at the beginning and end of the period Thus if the car in question was 1 5 years old at the beginning of the year and, of course, 2 5 years at the end of the year, it can be seen from the table that the depreciated value for such a car in Iowa would be 51 9 percent at the beginning of the year and 40 5 percent at the end of the year The depreciation for the year is the difference between the two or 11 4 percent of the value new If the delivered price was $700, the depreciation for the year would be 11 4 percent of $700 or $79 80

The interest charges for all cars were computed using a rate of 6 percent per annum on the depreciated value of the car at the beginning of the year or fraction of the year over which the summary was made

In the summary for the year the total cost for each item was obtained and this was then reduced to a unit cost, that is to the cost in cents per mile, by dividing the total cost by the total miles traveled during the year The total cost of operation for the car was then obtained by adding all of the individual costs A separate total was obtained for operation of the car not including extra help and also including extra help A third item reported was the cost of extra help without the car This covers the cost of extra help using a separate form of transportation required when the roads were impassable, and is referred to as car transportation replacement cost in this report

ANALYSIS OF COST RECORDS AND DISCUSSION OF RESULTS

Since only a small percentage of cars were reported as operating exclusively on one type of surface, it was necessary to devise methods of analyzing the cost records which would permit isolating the unit costs of operation on each surface as accurately as possible To accomplish this purpose two methods of analysis were used, one involved plotting the cost data on graph paper indicating the variations or trends in costs as the percentage of travel on earth, gravel, or on paved roads varied, the other involved the use of the statistical method of least squares to determine the unit costs on each type of surface There was substantial agreement in the results obtained by both of these methods in all cases in which the same basic data were used The graphical method, although lacking in accuracy, has the advantage of bringing to the reader a fairly complete picture of all of the facts related to cost and this is the method most frequently referred to in the following discussion

Comparison of Unit Costs for 160 Cars Using the Graphical Method

In Figure 5 the unit cost of operation for each of the 160 cars is shown for varying percentages of total mileage traveled on earth roads as compared to the mileage on pavement and untreated gravel for the year 1936 These costs represent the adjusted total operating costs and do not include the cost of extra help or a charge for the driver's time

TABLE 1

AVERAGE TIRE LIFE AND UNIT TIRE COSTS FOR VARYING AMOUNTS OF TRAVEL ON HARD SURFACED ROADS

Per cent hard surfaced roads	Average life per tire, miles	¢ per mi for four tires
0- 40	18,000	0 333333
40- 60	20,000	0 300000
60-100	22,000	0 272727

TABLE 2

DEPRECIATED VALUES OF IOWA AND INDIANA CARS

Age, years	Percent of value new*		Age, years	Percent of value new*	
	Iowa	Indiana		Iowa	Indiana
0 0	100 0	100 0	4 0	27 3	27 8
0 5	73 2	74 0	4 5	23 6	24 0
1 0	59 8	60 7	5 0	20 0	20 3
1 5	51 9	52 7	5 5	17 0	17 3
2 0	45 8	46 5	6 0	14 0	14 2
2 5	40 5	41 2	6 5	11 3	11 5
3 0	35 7	36 3	7 0	8 9	9 0
3 5	31 5	32 0	7 5	7 0	7 1

* Value new is the total delivered price paid by owner — 100%

It should be noted that the average cost for cars traveling exclusively on gravel and paving is about 4 cents per mile as compared to 8 cents per mile when the travel is exclusively on earth roads The minimum cost on gravel and paving is about 2½ cents per mile as compared to 6 cents per mile on earth and the

FORM 1

RURAL MAIL ROUTE CAR COST STUDY

ROUTE AND CAR DESCRIPTIONS

Nov 1, 1935

(1) Carrier David R Allred RR 5 Town Des Moines

(2) Street 1705 12 County Polk State Iowa

(3) Number of boxes on route 400.

(4) Miles traveled each day that route is carried 39.25

Road surface type	Route miles	Miles between home and P O (both ways)	Total daily miles
1 (Rigid pavements) Concrete, brick, sheet asphalt, wood block, granite block, etc	17	3	20
2 (Semi-rigid treated) Bituminous macadam, oil treated gravel, tarvia, etc			
3 (Surfaced but untreated) Gravel, crushed rock, sand-clay, shale, cinders, etc	17		17
4 (Natural earth) Clay, loam, gumbo, sandy soil, etc	2 25		2 25
Total	36 25	3	39 25

(5) Make of car Studebaker Year model 1935 Body style Sedan
No of cylinders 6 Date purchased 12 20-34 New or used New
List weight 2960 List price $785 (see license receipt)
Delivered price, new including equipment $986 50
Actual weight with driver and average mail load 3175
Monthly rental value of garage $3 00

(6) If you operate 2 cars, note similar information for the second car on the reverse side

Return to

Engineering Experiment Station
Iowa State College
Ames, Iowa

FORM 2

Page From Daily Record Book

Record for Feb 17 (Month and Day), 19 36

Speedometer reading, morning 18835

Approx air, temperature, morning 27°

Dry, wet, rainy, quiet, windy, icy, snowy
(check ones applicable)

Approx pounds of mail carried 75

Hours required to cover route 6

Miles driven on route 45

Itemized expenditures on this day

		Total Cost
Gasoline, gallons	8	$ 1 46
Oil, quarts	1	$ 0 26
Tire repair		$ 0 35
Chains		$ 5 90
Extra help		$ 1 00
		$

Hours personal labor on car 1

Remarks, road condition (impassable, slippery, √ soft, etc), delays, detours, accidents, extra help, √ chains used, √ etc

Road icy, one mile detour on earth

Miles driven off route by road surface types

Concrete, brick, asphalt, etc	Oiled gravel, asphalt or tarmacadam, etc	Plain gravel, shale, crushed rock, etc	Natural earth	Total
10		3	1	14

FORM 3

Tire Mileage and Cost Record

Serial No	Mfr or Make	Plies	Cost	Wheel	Put on		Taken off		Total mileage
					Date	Speedometer	Date	Speedometer	
730-K	Atlas	4	13 25	R F	8-14	29102			
960-K	"	"	"	L F	"	"			

FORM 4

Name David R Allred No 5
R R 5 No boxes 400 Town Des Moines County Polk State Iowa
Make Studebaker Year model 35 Body Sedan No cyls 6
Date purchased 12-20-34 New or used New List weight 2960
Road weight 3175 List price 785 Del cost 986 50 Garage rental $3 00

Surface type	Route	Home P O	Total daily
1 Pavement	17 00	3 00	20 00
2 Treated			
3 Gravel	17 00		17 00
4 Earth	2 25		2 25
Total	36 25	3 00	39 25

Cost Summary For Year

Item	Cost, $	¢ per mi
1 Gasoline		
2 Oil		
3 Tires		
4 Maintenance		
5 Depreciation		
6 License	20 00	
7 Garage		
8 Interest		
9 Insurance	30 00	
Total		
10 Extra help		
Total		

Miles per gallon, gas ______

Miles per quart, oil ______

Average speed on route ______

Month	Speedometer reading			Correct mileage	Surface mileage (Upper fig off route)			
	Beginning	End	Difference		1	2	3	4
Nov '35	16230	17627	1397	350 1047	342 534		450	8 63
Dec '35	17627	18833	1206	153 1053	153 537		453	63
Jan '36	18835	20004	1169	114 1055	114 538		454	63
Feb	20004	21203	1199	141 1058	141 540		455	63
Mar '36	21203	22664	1461	374 1087	374 554		467	66
Apr '36	22664	24097	1433	393 1040	393 530		447	63
May	24097	25580	1483	523 960	523 490		413	57
June	25580	26886	1306	423 883	423 450		380	53
July	26886	28396	1510	470 1040	470 530		447	63
Aug	28396	29850	1454	534 920	534 469		396	55
Sept	29850	31299	1449	449 1000	449 510		430	60
Oct	31299	32658	1359	285 1074	285 548		463	63
Nov	32658	34466	1808	1054 754	1054 385		324	45
Total								

FORM 5

Final Monthly Summary of All Operation Items

Name David R Allred No 5

Month	Total correct mileage route mi above	Gasoline Gal	Gasoline $	Gasoline Mi per gal	Oil Qts	Oil	Oil Mi per qt	Tire $	Maintenance, $ Chain an-fr	Maintenance, $ Wash Gr's	Maintenance, $ Rep	Maintenance, $ Lab 40¢	Maintenance, $ Total $	Maintenance, $ Total ¢ per mi	Total $	Total ¢ per mi	Extra help $	Av route speed	Remarks
Nov '35	1047 1397	110	20 12	12 7	10	3 26	140		4 05	0 75	0 87	2 80	8 47	0 606	31 85	2 280		12 9	Defroster— $1 53
Dec '35	1053 1206	99	17 12	12 2	10	3 28	121		0 35	1 50	1 00	1 60	4 45	0 369	24 85	2 061		11 2	
Jan '36	1055 1169	108	19 44	10 8	5	1 63	234		0 82 1 40	0 75	7 92	1 60	12 49	1 068	33 56	2 871	0 15	9 7	Storage— $0 75
Feb '36	1058 1199	113	20 79	8 9	10	3 25	120	0 50	0 31 1 35	0 75	3 80	2 40	8 61	0 718	33 15	2 765	2 00	9 2	Tow-chain— $2 39
Mar '36	1054 1440	123	21 92	11 7	5	1 63	288			2 40	2 72	2 80	7 92	0 550	31 47	2 185	1 50	10 0	
Apr	1040 1433	110	19 57	13 0	10	2 66	143			2 25	3 43	0 80	6 48	0 452	28 71	2 003		13 0	
May	960 1483	113	20 15	13 1	5	1 65	297			0 75	7 18	2 40	10 33	0 697	32 13	2 167		12 0	
June	883 1306	97	17 26	13 5	5	1 63	261			0 75	16 95	5 20	22 90	1 753	41 79	3 200		12 3	
July	1040 1510	100	17 78	15 1	10	3 26	151	0 50		2 50	1 00	1 60	5 10	0 338	26 64	1 764		12 9	Insurance 1 yr $30 00
Aug	920 1454	108	19 26	13 4	5	1 63	291	0 40 22 44		0 75	1 50	1 60	3 85	0 265	47 58	3 272		12 4	
Sept	1000 1449	111	19 71	13 1	10	3 26	145	0 85		0 75	1 52	2 00	4 27	0 295	28 09	1 939		12 2	
Oct	1074 1359	115	20 48	11 8	5	1 63	272	33 00	5 40	0 60 1 50	21 88	4 40	33 78	2 486	88 89	6 541		11 3	
Nov	754 1808	142	24 30	12 7	18	5 34	100			2 01	16 40	0 80	19 21	1 063	48 85	2 702		11 2	
Dec																			
Total																			

FORM 6

Final Quarterly and Annual Summary for All Car Operation Items

Name David R Allred R R 5 Town Des Moines County Polk State Iowa No boxes 400

Make Studebaker Director Year 1935 Body Sedan Road weight 3175 Date purchased 12-20-34 Delivered cost $986 50

Summary by Seasons

Period	Mileage			Mileage by surface types								Av* route speed	Gasoline		Oil		Maintenance		Extra help			
				Pavement		Treated		Gravel		Earth									With car		Without car	
	Total	Route	Off route	Route	Off route	Route	Off route	Route	Off route	Route	Off route	Mi per hr	Mi per gal	¢ per mi	Mi per qt	¢ per mi	$	¢ per mi	$	¢ per mi	$	¢ per mi
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)
Dec -Feb 1935-1936	3574	3166	408	1615	408			1362		189		10 0	11 2	1 605	143	0 228	25 55	0 715	2 15	0 068		
Mar -May 1936	4377	3087	1290	1574	1290			1327		186		11 6	12 6	1 408	219	0 136	24 73	0 565	1 50	0 049		
June-Aug 1936	4270	2843	1427	1449	1427			1223		171		12 6	14 0	1 272	214	0 153	31 85	0 746				
Sept -Nov 1936	4616	2828	1788	1443	1788			1217		168		11 6	12 6	1 397	140	0 222	57 26	1 240				
Year	16837	11924	4913	6081	4913			5129		714		11 3	12 6	1 412	172	0 183	139 39	0 828	3 65	0 031		

Summary for Year

		Gasoline	Oil	Tires	Maintenance	Garage	License and taces	Insurance	Depreciation	Interest	Total	Extra help†		Total‡
												With car	Without car	
		(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
Recorded cost	$	237 78	30 85	57 69	139 39	36 00	20 00	30 00			551 71	3 65		555 36
	¢ per mi	1 412	0 183	0 343	0 828	0 214	0 119	0 178			3 277	0 031		3 308
Adjusted cost	$	237 78	30 85	48 17	139 39	30 00	20 00	27 82	145 02	36 40	715 43	3 65		719 08
	¢ per mi	1 412	0 183	0 286	0 828	0 178	0 119	0 165	0 861	0 216	4 249	0 031		4 280

* Average route speed equals total route miles divided by the total time spent on the route

† Extra help is divided, where possible, into help used with the regular route car (recorded under "with car"), and help which is hired to cover the route or a portion of the route (recorded under "without car") Costs under "without car" are usually not true transportation costs as the salary of the operator is often included

‡ Column (14) is the sum of columns (11) and (12)

maximum is about 6¼ cents per mile on gravel and paving as compared to 11¼ cents per mile on earth One reason for this large spread in costs between the minimum and maximum values is due to the wide variations in annual mileage and the age or depreciation charges against the car To reduce the effect of the wide variations in mileage and age, the unit costs of 98 cars operated more than 5,000 miles per year and with ages of 3 years or less (1933 to 1936 year

The minimum cost remained the same but the maximum cost was raised about 1 to 1½ cents per mile The cost of extra help on gravel and paving is no doubt due almost entirely to blizzards and the presence of snow on these roads while on earth roads the cost of extra help was due not only to snow but also to the mud condition which interfered seriously with travel on these roads

The unit transportation replacement costs given in Figure 8 are quite variable

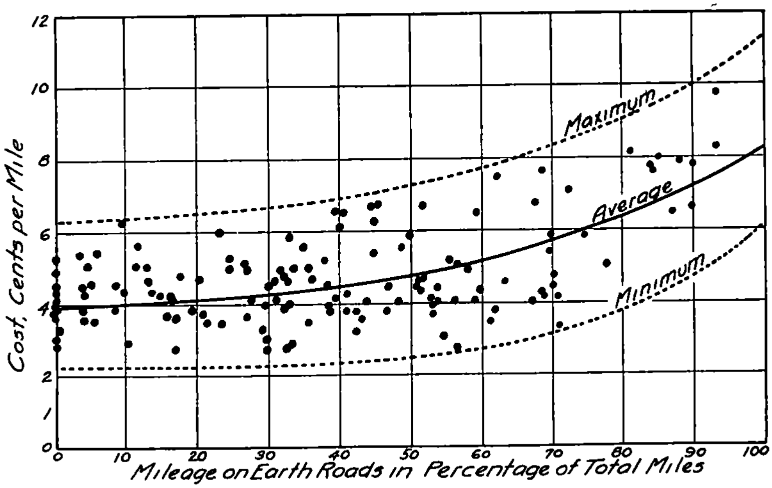

Figure 5 Unit Cost of Operation (Excluding Extra Help) for Varying Percentages of Total Mileage Traveled on Earth Roads as Compared to Mileage on Pavement and Untreated Gravel for the Year 1936

models) were plotted (Figure 6) and it is interesting to note that the average curve is almost identical to that obtained in Figure 5 It is to be expected that the differences between the maximum and minimum costs are less under these conditions than when the cost data for all cars are used This is borne out by the data in Figure 6

The item of extra help is shown in Figure 7 to have a marked effect on the cost for all cars for the year round condition although the effect is greater on earth roads than on gravel and paving On the former it raised the average cost only ½ cent per mile as compared to a 1½ cents per mile increase on the latter.

as might be expected The data are scattered so widely that an average curve probably should not be shown for these data Of course, since travel by horse or on foot is much slower than by car, the costs should normally be higher as indicated on the graph than under conditions when a car can be used Also, the small change in the average cost on gravel and paving as compared to earth indicated here should be expected because with roads impassible in either case the type and cost of transportation replacement would be very nearly the same on the average for both types of surface

The extreme seasonal effect is shown in Figure 9 in which the unit costs for

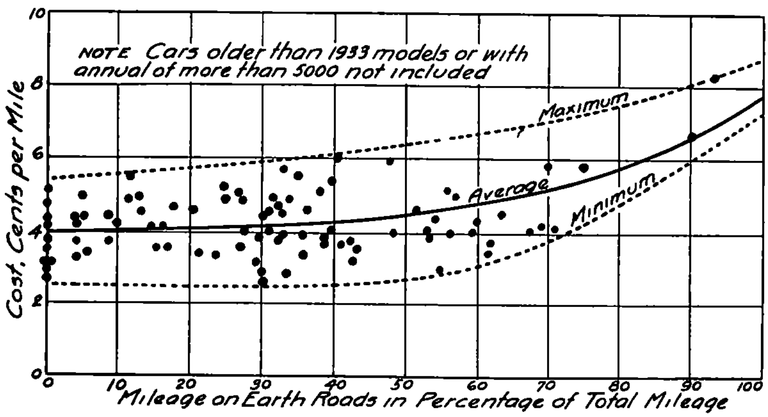

Figure 6 Unit Cost of Operation (Excluding Extra Help) for Varying Percentages of Total Mileage Traveled on Earth Roads as Compared to Mileage on Pavement and Untreated Gravel for the Year 1936.

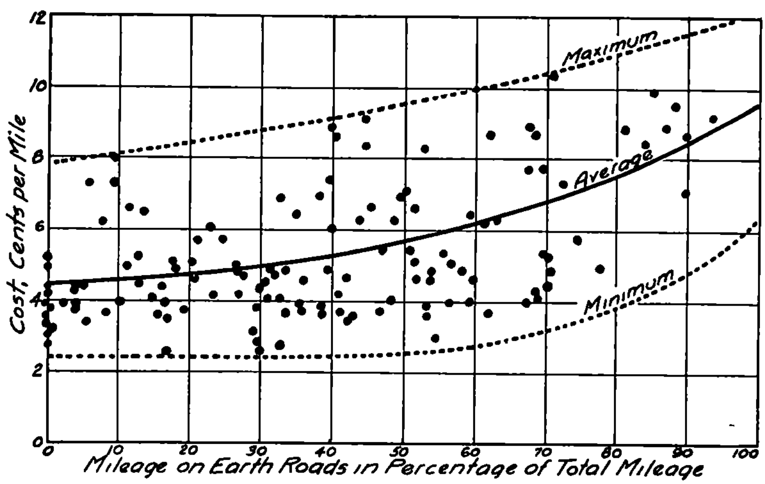

Figure 7 Unit Cost of Operation (Including Extra Help) for Varying Percentages of Total Mileage Traveled on Earth Roads as Compared to Mileage on Pavement and Untreated Gravel for the Year 1936

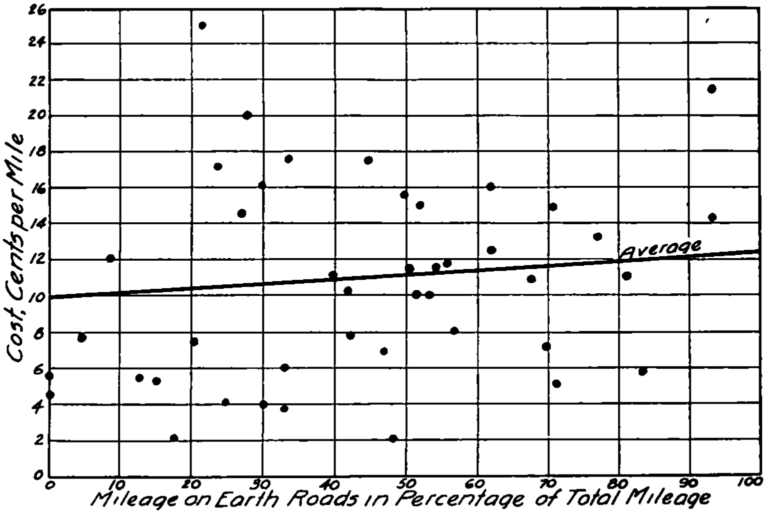

Figure 8 Unit Transportation Replacement Cost for Varying Percentages of Total Mileage Traveled by Regular Car on Earth Roads as Compared to Mileage on Pavement and Untreated Gravel

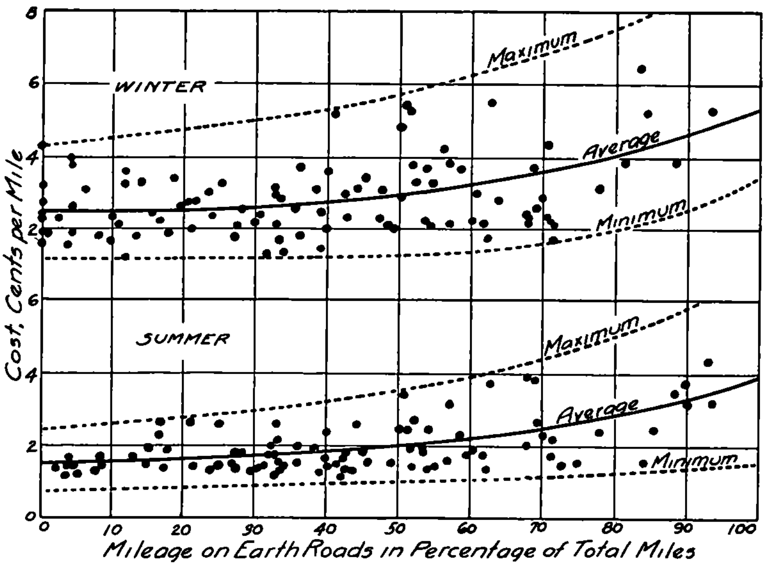

Figure 9 Unit Cost of Gas, Oil, and Maintenance for Varying Percentages of Total Mileage Traveled on Earth Roads as Compared to Mileage on Pavement and Untreated Gravel for (a) Winter of 1935-36, (b) Summer of 1936

gasoline, oil, and maintenance are given for all of the cars for the winter and summer seasons These curves show that the spread in unit costs is much greater in winter than in the summer season and on earth roads than on gravel and concrete In fact, on paved roads during the summer months the difference in the unit costs for these items of operation is roads The cost of gasoline, oil, maintenance and extra help on earth roads is just about double that on gravel or paved roads The curves in Figure 11 show that extra help is a factor only in winter and spring months on paved roads, whereas there is a definite extra cost indicated due to extra help during all seasons for cars operating on earth roads

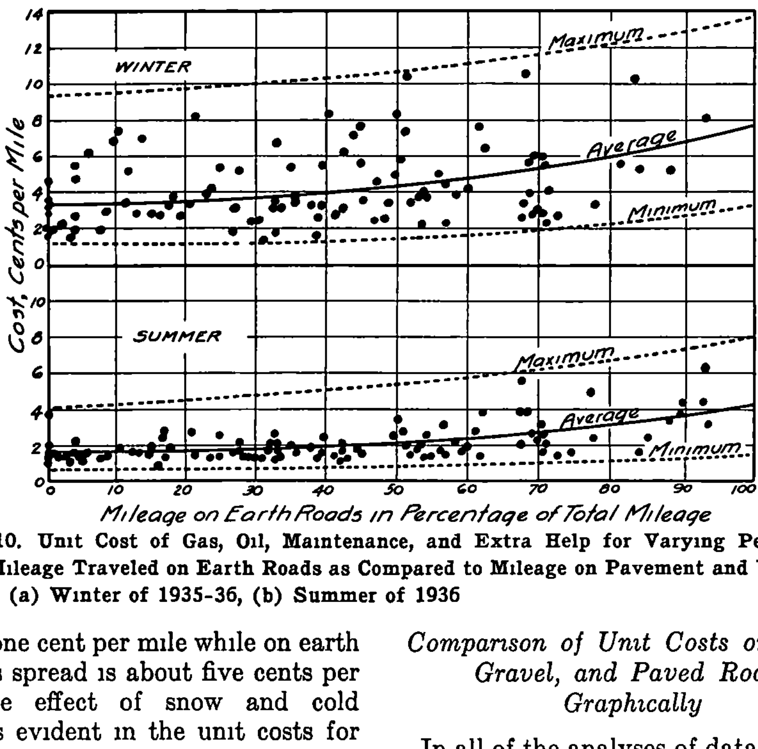

Figure 10. Unit Cost of Gas, Oil, Maintenance, and Extra Help for Varying Percentages of Total Mileage Traveled on Earth Roads as Compared to Mileage on Pavement and Untreated Gravel for (a) Winter of 1935-36, (b) Summer of 1936

less than one cent per mile while on earth roads, this spread is about five cents per mile The effect of snow and cold weather is evident in the unit costs for the winter months, the average cost on paving being about one cent higher in winter than in summer This represents an increase of about 70 percent due to the extreme seasonal effect These figures give some indication of the extent to which traffic profits by snow removal

When extra help is included in the items of cost to bring out the seasonal effect as shown in Figure 10, there is practically no change noted in the costs for the summer months but there is an average increase of about one cent per mile on gravel and paving and slightly more than three cents per mile on earth

Comparison of Unit Costs on Earth, Gravel, and Paved Roads Graphically

In all of the analyses of data discussed up to this point, the comparisons were made between the costs on earth roads as compared with the costs on gravel and concrete combined This comparison seemed fair because the data indicated that the greatest differences in cost were those due to travel on mud roads, the differences between gravel and paved roads being very much smaller The number of samples available to make a study of the direct comparison between the costs on earth and gravel, earth and paving, and gravel and paving seem rather small to establish definite trends,

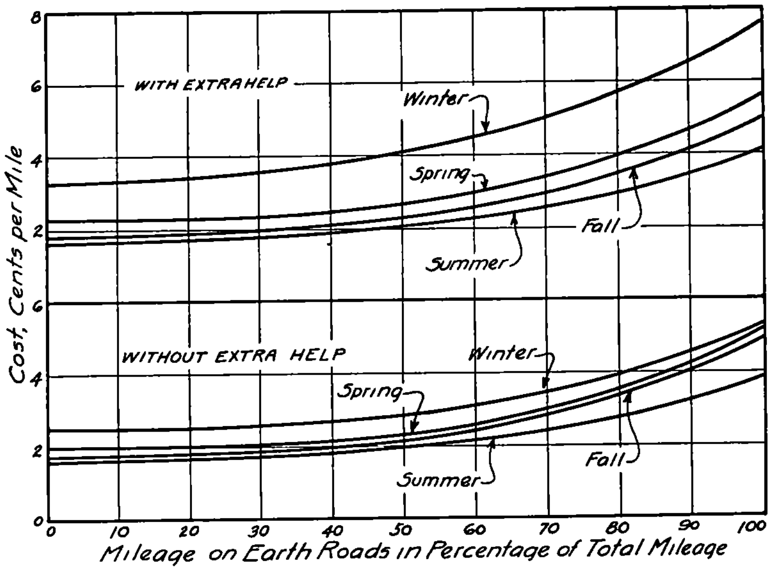

Figure 11 Unit Cost of Gas, Oil, and Maintenance for Varying Percentages of Total Mileage Traveled on Earth Roads as Compared to Mileage on Pavement and Untreated Gravel for Seasons of 1936 (a) With Extra Help, and (b) Without Extra Help

nevertheless, these data are shown in Figure 12 and seem to be fairly consistent with the data previously discussed The greatest difference in the unit cost of operation, according to these data, is that between gravel and earth which amounts to about $4\frac{3}{4}$ cents per mile In the comparison between earth and paving this difference is reduced to $3\frac{1}{2}$ cents per mile But if the unit cost on earth of $8\frac{3}{4}$ cents per mile, shown in the comparison between earth and gravel, is used as the correct cost, the difference between the cost on paving and earth is about five cents per mile The average difference in cost on gravel and paving is shown to be about $1\frac{1}{4}$ cents per mile, the average cost on paving being about $3\frac{1}{2}$ cents per mile as compared to $4\frac{3}{4}$ cents per mile on gravel for the year round condition and not including extra help

Using the data for these same cars to study the seasonal effect, it is interesting to note that the greatest spread in unit costs for gas, oil, and maintenance

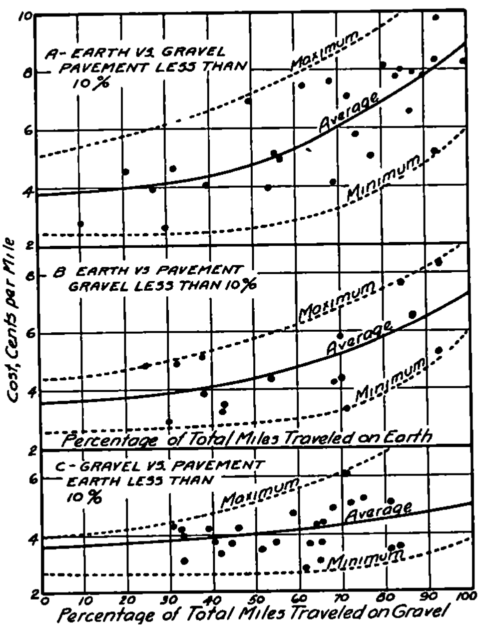

Figure 12 Unit Cost of Operation (Excluding Extra Help) on (a) earth as compared to Gravel, (b) on Earth as Compared to Pavement, and (c) on Gravel as Compared to Pavement

is obtained in the comparison between earth and gravel and the smallest difference is shown in the comparison between gravel and paving It is significant that the cost of gasoline, oil, and maintenance is the same for all the percentages of gravel during the summer months when comparing costs on gravel versus costs on paving In other words these data indicate that the difference in cost on gravel and on paving is very slight during the summer months

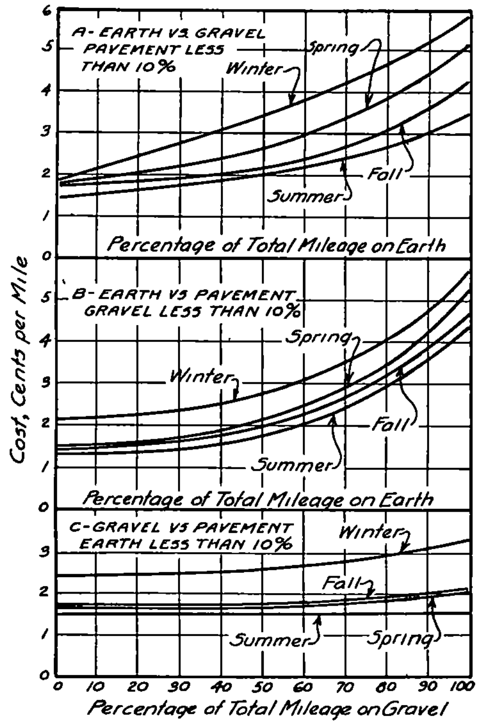

Figure 13 A Comparison of the Unit Costs of Gas, Oil, and Maintenance for the Four Seasons on (a) Earth vs Gravel, (b) Earth vs. Pavement, and (c) Gravel vs Pavement.

Annual Mileage of Cars

In the operation of any car, there are certain total annual charges such as garage rental, license fee, taxes, insurance, interest and depreciation which are the same or very nearly so, no matter what mileage the car is driven during the year Variations in annual mileage will, therefore, bring about corresponding variations in the total unit costs of operation of the car A car which is driven a relatively low mileage during the year will show a correspondingly large increase in the unit cost of operation, whereas for a large annual mileage the unit costs will show a corresponding decrease The data in Figure 14 indicate a wide variation in the annual mileage The greatest annual mileage is shown to

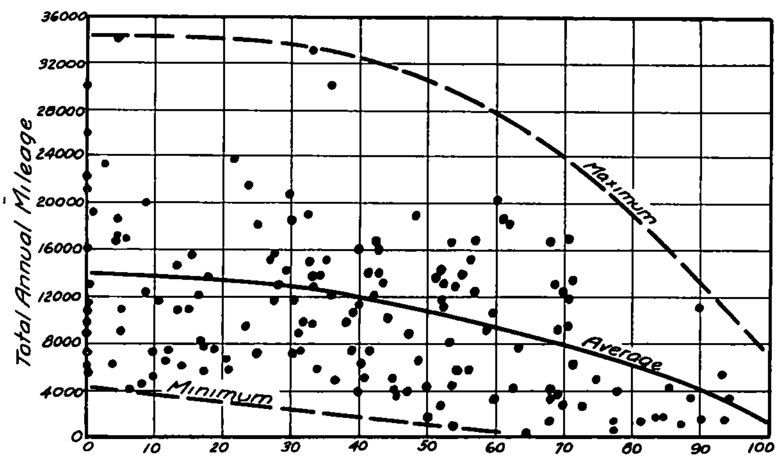

Figure 14 Annual Mileages for 160 Cars for Varying Percentages of Total Mileage Traveled on Earth Roads as Compared to Mileage on Pavement and Untreated Gravel

be driven on gravel and paving and the lowest when the operation is exclusively on earth roads By taking the actual mileage records it was found that the total mileage driven by the 160 cars was 1,747,040 miles or an average of 10,919 miles per car Of this average 3,516 miles were on paving, 3,789 on gravel, and 3,614 on earth roads However, taking the trends as indicated by the average curve in Figure 14, the average annual mileage of a car operated exclusively on earth roads was only about 1,500 miles, on paving 14,000 miles, and on gravel a separate study indicated the mileage to be about 11,000 miles annually

The differences in the annual mileages on earth, paving, and gravel account in part for the large differences in the unit cost of operation The graphical data show that the difference in the total unit cost on gravel or paving and on earth is about 4¼ cents per mile whereas the difference in the cost of fuel, oil, and maintenance is only about 2½ cents per mile This 1¾-cent increase shown by the average curve is due to the low annual mileage on earth as compared to the mileage on gravel or paving, and to the higher average age of cars traveling on earth as compared to gravel and paving

Age of Cars

The ages of the cars driven over the various types of roads had a variable effect on the unit cost of transportation because a higher rate of depreciation is charged against new cars than against older cars From the records in Form 1, the average age of all cars was found to be 2 years and 3 months The average age of cars operated on paving and gravel was found to be 1 year and 1 month and for cars operated on earth roads 4 years and 3 months Since the rate of depreciation is much lower for cars operated on earth, it will be seen that the higher unit cost on earth road due to the low annual mileage on earth will be offset in part by the lower unit cost for the item of depreciation While it is quite logical to expect that older cars should be driven on earth roads and that the annual mileages on earth should be lower than on gravel or paving, it seems fair to state that in comparing unit costs greater weight should probably be given to the fact that the annual mileage may be low on earth roads than that the average age may be high because under certain weather conditions it is very difficult to travel large mileages whereas the age of the car is not so affected

Average Rate of Travel on the Route

While the rate of travel did not enter directly into the unit cost computations, it is an important factor which should not be overlooked The rate of travel on the route is shown in Figure 15 The average rate of travel should not be confused with the average speed because the former is based on the entire time required to cover the route including stops and does not give a direct indication of the average road speed The average curve for the rate of travel shows a definite rate of decrease as the mileage on earth is increased The average rate is about 4 miles an hour slower in winter than during the summer months The average rate of travel on earth is about 8 miles per hour as compared to 14 miles per hour on gravel and paving

The most significant use which can be made of these data is that these differences in the rates of travel represent a real time saving value to the mail carriers Thus, if the low value of 50 cents per hour were to be used to represent the value of the average mail carrier's time, then for these particular data the carriers operating on gravel or paving would have a time saving advantage equal to 2⅝ cents per mile A 60 mile route on earth would require 7½ hours on an average day as compared to about 4¼ hours to cover the same distance on gravel or paving For differences as large as this there can be little doubt but that the time saving factor provides a real economic advantage for the gravel or paved road over the earth road for operations of the mail carrier type

which were used in obtaining adjusted costs in the graphical solution

Using the average annual mileage of 10,919 miles and the standard fixed charges or overhead costs previously established, the total unit cost of operation for the year round condition will then be 4 82 cents per mile on earth, 4 11 cents per mile on gravel, and 3 38 cents per mile on paving If the average age of the cars on earth is increased to 4

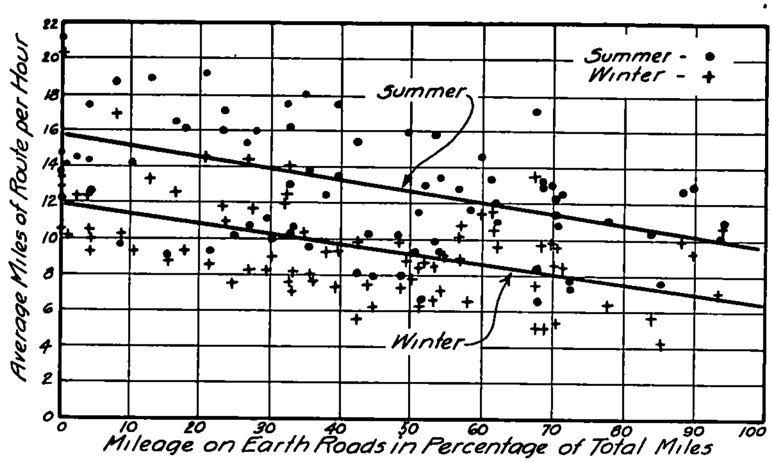

Figure 15 Average Miles of Route Covered Per Hour, Including Stops, for Varying Percentages of Total Mileage Traveled on Earth Roads as Compared to Mileage on Pavement and Untreated Gravel Surfaces

Statistical Analysis by Method of Least Squares

The comparative costs of gasoline, oil, and maintenance for earth, gravel, and concrete were computed by the statistical method of least squares to be 3 17 cents per mile, 2 46 cents per mile, and 1 73 cents respectively for the entire year, 3 44 cents per mile, 2 56 cents per mile, and 2 06 cents per mile for the winter months, and 1 97 cents per mile, 1 71 cents per mile, and 1 49 cents per mile respectively, for the summer months These results are based on the actual recorded costs and are not adjusted for mileage, age, or any of the other factors

years and 3 months and the annual mileage reduced to 1,500 miles per year, the total unit costs on earth will then be 9 23 cents per mile, which is slightly higher than the cost obtained in the graphical solution The unit cost of operation on earth might quite properly be further increased by about ¾ cents per mile for extra help and 2⅝ cents per mile as a time factor representing the money value of time saved on gravel or paving over travel on earth for this particular group of drivers The extreme average difference in the cost of operation on earth as compared to paving is therefore about 9½ cents per mile for the year round conditions, while a conservative difference is $1\frac{1}{2}$ cents per mile if only the savings in gasoline, oil, tires and maintenance are considered

APPLICATION OF UNIT OPERATING COST DATA TO A STUDY OF ECONOMIC COMPARISONS OF EARTH AND GRAVEL ROAD SURFACES

The purpose of determining the unit operating costs for various road surfaces is mainly to make it possible for the engineer to select the types of surface construction which will furnish transportation at the lowest cost The amount of money which the engineer is justified in spending for road surface construction depends quite largely on the volume and type of traffic operating on the road If the traffic is light, the road will of necessity be a low cost type but if the traffic exceeds 100 or 200 vehicles a day a more costly or higher type of construction will generally be necessary to provide the most satisfactory transportation service at the lowest cost The solution of the problem of determining the traffic volumes required before the change from one surface type to a higher surface type is justified is not an easy one because reliable and accurate cost data have not been available Since the data in this report have provided for the first time some definite information concerning the detailed costs of operation on unimproved earth roads as compared to gravel roads, a general solution of this problem to indicate when the engineer is justified to improve an earth road with a gravel surface seems to be an appropriate part of this report The following solution applies particularly to roads in the middlewest on which the rural mail, the school bus, and similar types of service requiring daily operation in all kinds of weather, constitute the bulk of the traffic

In making the economic comparison of earth and gravel roads, the total annual transportation cost per mile of road for varying volumes of traffic was computed using the following formula

Total annual transportation cost per mile of road = Annual maintenance cost per mile + annual depreciation per mile + annual interest per mile + annual motor vehicle operating cost per mile

Although the maintenance and depreciation costs for various surface types vary depending on the volume of traffic, the traffic in this case will be assumed to be so light (from 0 to 50 vehicles a day) that the differences in maintenance and depreciation can be neglected Recent annual reports of county engineers in Iowa have indicated that $75 00 per mile is a fair average maintenance cost for local county earth roads and $100 00 per mile for gravel roads This latter charge may be assumed to include an allowance for gravel replacement to keep the surface perpetually as good as or better than new In Iowa it is generally true that county gravel roads require less blading or similar maintenance than do earth roads to keep them in a smooth serviceable condition

For the natural earth surface no charge for depreciation or interest is necessary In the case of the gravel road the depreciation charge is included in the maintenance charge An interest rate of 4 percent was charged on the investment which for this solution was assumed to be $1,000 00 per mile Many Iowa local county roads have been gravel surfaced for considerably less than $1,000 00 per mile, but this seems to be a fair and convenient figure to use in solving this problem While a lower interest charge, or in fact no interest charge, might be recommended by some engineers, the result would be changed only slightly The higher interest charge for the larger investment requires more traffic to justify the investment and thus is on the conservative side in reaching a decision

In making the economic comparisons shown in Figure 16, three different values of unit operating cost were used. In Figure 16a the same average annual mileage and the same average age was used for cars operating on gravel as for

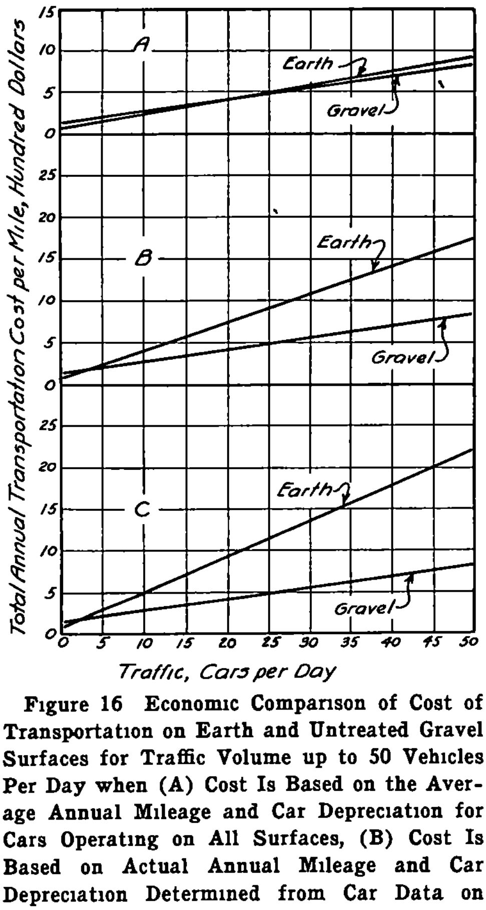

Figure 16. Economic Comparison of Cost of Transportation on Earth and Untreated Gravel Surfaces for Traffic Volume up to 50 Vehicles Per Day when (A) Cost Is Based on the Average Annual Mileage and Car Depreciation for Cars Operating on All Surfaces, (B) Cost Is Based on Actual Annual Mileage and Car Depreciation Determined from Car Data on Each Surface, and (C) Cost Is Based on Actual Annual Mileage and Car Depreciation and Including also a Time Factor for Slower Rates of Travel on Earth

the cars operating on earth. The unit cost on earth in this case was 4.82 cents per mile and on gravel 4.11 cents per mile, the higher cost being due to increased gasoline, oil, tire and maintenance costs on earth than on gravel. In Figure 16b the cost on earth was increased to 9.23 cents per mile which represents an adjusted cost based on the lower annual mileage, the greater average age, and the higher cost of extra help on earth than on gravel as determined from the mail carrier cost records. In Figure 16c the cost on earth was further increased to 11.90 cents per mile, the additional increase being the 2⅔ cents per mile time factor which it was shown is the value of time saved by traveling on gravel instead of earth if the driver's time is assumed to be worth 50 cents per hour.

The curves in Figure 16c show that a traffic volume of 25 cars per day will provide sufficient savings in gasoline, oil, tires, and maintenance when operating on gravel instead of earth roads, to justify the $1,000 investment and the $25.00 per year extra maintenance required to build and maintain the gravel road. It should be noted that the small difference of 0.7 cents per mile in operating costs on gravel and earth is sufficient to justify this improvement with only 25 vehicles per day. If the total costs of operation on earth and gravel as determined in this study are used, the curves in Figure 16b indicate that a traffic volume of 4 vehicles per day justify the change from earth to gravel. And if the value of time saving is added, a traffic volume of 3 vehicles per day will justify the change.

Now, of course, anyone familiar with studies in economic comparisons will know that while savings of one cent per mile and of 5½ or 8 cents a mile will justify the changes indicated above, there is no actual saving unless the traffic volumes are greater than those indicated or unless additional benefits, which cannot be evaluated, result from the changes. That the latter is generally accepted to be true is conceded by persons familiar with this situation.

There is another very real and very practical question which must be met, if the findings of such studies as this are to be put into effect That is, although the engineer can show that savings in operation cost of 1 cent per mile or 5½ or 8 cents per mile are possible, he still is confronted with the problem of collecting the 1 cent per mile or the 5½ or 8 cents per mile tax to pay for the improvements which make these savings possible The uninformed car owner in the rural areas in many cases will prefer to pay a "mud tax" in small amounts daily or weekly than to pay the large road tax, gas tax, or license fee needed for road improvements which will bring returns far exceeding the tax charged against the car owner The average motor vehicle tax in Iowa and in the United States as a whole is easily less than ½ cent per mile and yet the savings or benefits derived from this tax, or service charge as it more properly should be called, far exceeds the ½ cent tax per mile It is doubtful if a satisfactory tax plan can be devised which will make it possible to collect a use tax or vehicle tax as large as 1 cent per mile and certainly not as large as 5½ cents or 8 cents per mile The fact still remains that unless a satisfactory plan can be found for financing road improvements of this type, car owners will continue to pay a "mud tax" on earth roads They will pay for road improvements which were never built Nor will car owners living along side of earth roads enjoy the fire protection, the health protection, and the social, educational, marketing and other advantages which follow in the train of road improvements in which the rural home is made accessible in all kinds of weather

In the mid west states, the secondary road improvement situation has not yet reached the stage where the engineer is concerned with improvements justified by a traffic of 5 vehicles per day, but there are thousands of miles of roads in Iowa alone on which the traffic ranges from 20 to 50 or more vehicles per day for which basic information of the type sought in this kind of study will be very useful in selecting surface improvements and maintenance methods providing satisfactory transportation service at the lowest cost

SUMMARY AND CONCLUSIONS

While this report is intended as a progress report and the work is to be continued, it has progressed far enough to warrant the presentation of the following summary and conclusions

1 The average annual mileage for the 160 mail carrier cars was 10,919 miles, thereby providing 1,747,040 miles of travel for which complete cost records were available for study

2 From the plotted data, the average cost of operation for the year round condition was 8 cents per mile on earth, 5 cents per mile on gravel, and 3½ cents per mile on paving

3 The average operating cost was 2¼ cents per mile less in summer than in winter for earth roads, 1¾ cents per mile less for gravel, and 1 cent per mile less for paving

4 Expenditures for mud elimination and snow removal are amply justified if the traffic volume exceeds 25 vehicles per day

5 The plotted data indicated an average annual mileage of 14,000 miles per year on paving, 11,000 miles per year on gravel, and 1,500 miles per year on earth

6 The average miles of route covered per hour, including stops, was 14 on gravel and paving and 8 on earth roads for the year round condition During the summer it was 16 miles per hour on paving and gravel and 10 miles per hour on earth During the winter months, it was 12 miles per hour on paving and gravel as compared to 6½ miles per hour on earth

7 The time value of paved or gravel roads over earth roads for these data is

2⅔ cents per mile if the carrier's time is worth 50 cents per hour

8 Extra help increased the average cost of operation on gravel and paving ½ cent per mile and on earth 1¼ cents per mile

9 The average transportation replacement cost (for horses, teams, or men on foot) was from 50 to 100 percent higher when the roads were impassable than the average car operating cost when the roads were passable

10 The average age of the 160 cars was 2 years and 3 months The average age of cars operated predominantly on earth was 4 years and 3 months and on gravel and paving 1 year and 1 month.

11 In the statistical analysis using the method of least squares, the average cost of gasoline, oil, tires, and maintenance for the year was 3 17 cents per mile on earth, 2 46 cents per mile on gravel, and 1 73 cents per mile on paving

12 The cost of gasoline, oil, and maintenance for the winter months was 3 44 cents per mile on earth, 2 56 cents per mile on gravel and 2 06 cents per mile on paving, while for the summer months it was 1 97 cents per mile, 1 71 cents per mile, and 1 49 cents per mile, respectively

13 A traffic volume of 25 vehicles per day will justify an investment of $1,000 00 per mile and an extra annual maintenance expenditure of $25 00 per mile in improving an earth road with a gravel surface based on the saving of 0 7 cents per mile in operating cost on gravel as compared to earth indicated in the cost records of the 160 cars studied in this report

14 A traffic volume of 3 vehicles per day will justify the improvement from a natural earth to a gravel surface if the annual mileage factor, the average age of car, extra help and the time factor are evaluated according to the cost data of these particular cars

15 The cost data indicate that a radical change in our highway taxation and financial policies for secondary road improvement is needed if it is desired to eliminate the "mud tax" which persists on our local county and farm-to-market roads

ACKNOWLEDGMENTS

The preliminary work on this project was carried on largely under the direction of Robley Winfrey, Research Engineer, for the Iowa Engineering Experiment Station By the time the project was actually under way and cost records were being sent in, Mr Winfrey was on leave with the Bureau of Public Roads, and the supervision of the project was turned over to the writer

Much of the detailed work in connection with the assembling and summarizing of the cost data has been under the supervision of Edwin R Davis, Louis W Heichenroeder, and Clarence W Rice, research assistants of the Experiment Station A large portion of the detailed office work was done by a staff of N Y A student workers

Special thanks are due the rural mail carriers who cooperated in every way possible to provide us with the information and cost data which have made this report possible

DISCUSSION ON OPERATING COSTS

Prof R L Morrison, *University of Michigan* I believe that the widest possible publicity should be given to direct savings to motorists because of highway improvement The motor vehicle owner is continually being told what large taxes he is paying but very little is said of the savings resulting from the road improvements which the taxes make possible If it costs 1½ cents per mile more to travel on gravel than on a pavement, then the gravel road collects a tax equivalent to about 22 cents per gallon of gasoline, or more than three times as much as any state collects In other words, assuming that he gets 15 miles per gallon of gas, for every 15 miles he travels on a pavement the motorist pays from 2 to 7 cents gas tax to the state, depending upon what state he is in, but for 15 miles on a gravel road he also pays an additional 22 cents, or more, for added gasoline consumption, tire wear, etc

MR FRED LAVIS, *Consulting Engineer:* I can only emphasize what Professor Morrison has said about the difficulties of getting people generally, and even some so-called experts, to appreciate the importance of these cost studies It is probably not necessary to preach this doctrine to this audience as I expect you all appreciate it I do think, however, that it is desirable not only that we ourselves realize the economic importance of this matter but that we should also disseminate the knowledge and preach its importance to laymen in such a way that it will be appreciated by them also

One of the first studies of this kind was made in connection with the layout and design of what is now Route 25 New Jersey, between the Holland Tunnel and Elizabeth—one of the earlier so-called super-highways There we attempted to evaluate the effect of the location, of rise and fall, rates of gradient, curvature, loss of time, etc, on the operating costs of vehicles The general public, however, was not interested in this The talk about saving a cent a vehicle mile, or that the loss of a minute in time was valued at a tenth of a cent, or what ever the values may have been, did not appeal to them Even when multiplied by 10,000,000 or 15,000,000 vehicles a year it was thought to be a purely theoretical engineering calculation which did not affect any individual They could not visualize the losses of these small sums of money

In speaking at various public meetings, however, I recall two illustrations which seemed to be appreciated This was before the construction of the Holland Tunnel and the ferries between New Jersey and New York were always congested at week-ends and holidays There were often long queues of vehicles waiting to get on the ferries and I suggested that if a man in a car were waiting at the rear end of one of these long lines, and someone offered for, say, half a dollar, to let him go forward to the front of the line, he would gladly have paid the amount His time or his convenience, or his pleasure was, therefore, worth something—it had some money value

At another meeting in the City of Newark we were explaining that the construction of this by-pass highway would relieve the congestion in the streets of that city and, among other things, permit easier access to the stores for shopping, etc The meeting had been addressed by representatives of various social societies, the Elks, Lions Clubs, etc, when a farmer from out of town arose and said, while I don't belong to none of these yere animal clubs, I want to say it would be a fine thing if some of us could get here in town without its taking a whole day to buy a shirt

Not long ago a Professor in one of our universities, who has a good deal to do with teaching transportation, expressed some skepticism over these "fine spun theories" but if they are applied with reason, they may be a very valuable guide in the economic design of some of our more important highways, costing $500,000 a mile and more, which affects for many years the operating costs of many millions of vehicles.

MR T W NORCROSS· *U S Forest Service* The importance of reductions in the cost of vehicle operation should not be minimized But there are other values and savings which are infrequently mentioned and which are of real importance For instance the value of the savings in travel time resulting from traveling over a better and higher standard road This saving, especially on a business road, frequently greatly exceeds the savings in operation cost of vehicles In fact for many classes of road use, the travel time is a good measure of the service value of the road But there are other savings resulting from better transportation and these cannot be adequately evaluated by the amount or speed of traffic These arise through the service rendered to the protection, development and utilization of resources—agriculture, mining, power, timber, etc I will give one illustration Up until the past few years the regular established practice of stockmen at the end of the grazing period was to drive the sheep over a road or stock driveway to the market or railway shipping point Not only were the services of several herders required but the time required was great, the sheep lost much weight during their arduous and slow journey of several days, feed along the route was seriously reduced, road banks were broken down, ditches and culverts clogged and finally when the journey was finished, it frequently developed that the market price was not as favorable as a few days earlier Now, to a greater and greater extent, trucks are being used The stockman determines the market price by radio, phone or newspaper When he is satisfied that it is the right time for him to sell, the sheep are loaded into trucks and within a few hours are delivered in prime condition at the market or shipping point Measured by volume of traffic, the service value of the road is of little consequence But measured by the money savings of each truck load, the road is of high value

SAFETY AND SPEEDS AS AFFECTING HIGHWAY DESIGN

By Fred Lavis
Consulting Engineer

SYNOPSIS

There has been considerable agitation in recent years for highways designed to permit "safe driving" at high speeds It is claimed by some that much of our main road system is obsolete, and that we need to build safety into the highways to provide adequate accommodations for modern high speed traffic Speeds of 90 to 100 m p.h today and even higher speeds in the future are advocated by certain engineers as proper bases for such design In discussing these proposals it is pointed out by the author that there are other road requirements than those of the superhighway, and that available funds must be distributed equitably among all classes of highways Moreover there may be a net loss in excessive vehicle operating costs resulting from high-speed travel, and traffic accidents are known to be numerous on the so-called "safe" straightaway stretches where higher speeds are possible It is felt that none but a few very experienced drivers are really capable of driving safely at speeds over 50 to 60 m p h , and that the education and selection of drivers is of far greater importance than the provision of speedways One of the greatest problems now confronting highway officials is the relief of traffic congestion, and maximum capacity on highways of heavy traffic is attained at speeds less than 50 m p h Although the laying out of highways should include all the improvements of design economically feasible, there appears to be no justification for designing high-speed roads when such design will entail additional expenditures for construction It is inconceivable that any considerable volume of traffic will move over a highway at speeds in excess of 50 to 60 m p h

Many engineers and some laymen have advocated, particularly during the past year or two, the need for designing highways to permit driving motor vehicles at higher speeds, or, as it is frequently expressed, "to drive safely" at these higher speeds Speeds of 90 to 100 m p h today, and even higher speeds in the not distant future, are advocated as proper bases of such design At the same time there have also been increasingly frequent notices and discussions of the excessively large numbers of highway accidents and fatalities, and it has been stated by many writers, in the technical press, periodicals and daily newspapers, that blame for such accidents may be laid in no small part to the highway structure

The (highway) Accident Commission of the Department of Commerce of the United States, however, in a report to Secretary Roper, says (New York *Times*, Feb 18, 1937), "The outstanding cause is high speed Road surface conditions play only a minor part "

At the meeting of the Highway Research Board in November, 1936, the very pertinent question was asked by Prof R A Moyer,[1] "What are the speed standards which should be adopted in the design of the various classes of highways of the future?" Upon the answer to this question, he said, centers much of the safety of our highways and the extent to which funds for highway construction may be used most effectively

It is not infrequently stated that (because of speed limitations) highway engineering practice is not keeping abreast of motor vehicle design and present traffic requirements, that highways are in many cases obsolete even before their paved surfaces have begun to show signs of wear It is doubtful, however, if in the present state of the art in this country,

[1] *Proceedings*, Highway Research Board, Vol 16, p 80 (1936)

with its vast network of all kinds of roads, all clamoring for improvement, where the construction and maintenance problem is so large and only in the earliest stages, and where all parts have to be kept going while improvements are being carried on, that any highway can properly be called obsolete

The alleged obsolescence is said to be due to

Inadequate widths of roadway or rights of way
Insufficient number or widths of traffic lanes
Need of firm, wide shoulders.
Obstruction of paved roadways by parked, stopped or slow moving vehicles (trucks on heavy gradients)
Crossings of other highways or railways at grade, crossings of lines of travel
Inadequate (obstructed) sight distances and other factors which would, if improved, facilitate driving at the high speeds of which modern cars are capable
Inadequate recognition of the speed possibilities of the modern automobile
Insufficient super-elevation of curves.
Inadequate lighting at night

All this boils down to the idea, in the minds of those who talk about obsolescence, that every highway which does not afford clear uninterrupted smooth surfaces permitting automobiles to be driven at 100 m p h is obsolete, and the question is thus presented as to whether or not engineers are lacking in adequate appreciation of all the factors which should govern design

DIMENSIONS OF THE PROBLEM

Prof R A Moyer has pointed out[2] that the speed factor affects or may affect such details of design as curvature, super-elevation, road widths, and sight distances, and so practically the whole design of first class highways

The question is important because of its effect on the expenditure of large sums of money Highway expenditures in the United States now amount to a billion and a quarter dollars annually on State highways and one billion on county and urban highways

It therefore seems desirable to review these questions and the arguments presented by recent writers and commentators The main questions which arise are

Is engineering design abreast of modern highway traffic demands?
To what extent is inadequate and improper design responsible for highway accidents?
Are many of our recently constructed highways obsolete, and what constitutes obsolescence?
To what extent should highway revenues be used for special facilities and for the few rather than for the extension of the net of reasonably good roads for general use?
What economic or other justification is there for considering speeds in excess of, say, 50 or 60 m p h as a factor in design?

NEED OF CAREFUL AND COMPETENT STUDY

There can be no question, of course, as to the necessity and desirability of the most careful study of traffic requirements, both present and future, as well as of the effect of the design on ease and facility of traffic movement, and on the operation of vehicles, but there must also be recognition of the fact that it is not always possible to provide funds for expensive types of construction The designer, therefore, while remembering that he is building today a structure to meet traffic requirements and necessities for some years to come, not only has to recognize the limitations of available funds but

[2] Proceedings Am Soc C E, May, 1937, p 941

also the economic relation governing the cost, original cost plus interest and the deterioration due to time, of constructing some facilities today which may not actually be needed for several years Competent engineers and designers also will not fail to take account of the desirability of providing for certain basic requirements, at the moment, while perhaps leaving the actual construction of certain parts of the structure to the future, and they will also recognize and properly balance the requirements of all users of the highways Some of the recent criticisms are

CRITICISMS OF MODERN HIGHWAY DESIGN AND CONSTRUCTION

The New York *Times* of December 6, 1936 carried an article based on a press release of the Regional Plan Association (of New York) with head lines stating "German highways held up as model" and "Regional Plan group reports Reich progress surpasses this Country" The press release read in part

> "Germany is building 5,000 miles of superhighways designed to permit safe automobile driving at a speed of 112 miles per hour . This provision for fast motor traffic . is contrasted with modern road building in the United States where there are ' no highways over which half the speed contemplated by the German engineers can be maintained in safety Germany has found a way to adapt her highways to the exigencies of national defense while in this country we have not yet served the purposes of peace"

The statement does not explain that Germany finds it necessary to provide for the exigencies of contemplated war even at the expense of the amenities of peace, or what is meant by "maintaining" this speed (112 m p h) *"in safety"* but it may be assumed to mean that the physical structure and condition of the highway permit this No mention is made of the effect of density of traffic on this speed and the "Regional Plan" in its statement disclaims the advocacy of such high speeds as desirable in the United States It seemed, however, to be the intention to convey through this article the idea that American engineers or highway authorities are behindhand in their conceptions of such structures The form of the structure, as now being built in Germany (Reichautobahnen), however, is not ahead of our ideas in this country As a matter of fact the basic principle was developed here 25 years ago It must be remembered, however, that this is a purely military undertaking by the Germans and has, so far as can now be judged, little if any economic justification, unless provision for war is such justification The construction of these highways is understood to be part of the German unemployment relief program

H A Phillips, writing of these highways in "Travel" of June 1937 says, "By-passing of towns and villages is essential because Germany's smaller and older towns have narrow and winding streets . Curves are widened and well saucered so that they may be rounded at a speed of 60 m p h" There is here some discrepancy with the statement that these roads are designed for speeds of 112 m p h

In a paper entitled "The Modern Express Highway" recently presented to the American Society of Civil Engineers (Proceedings, September, 1936), Mr Charles M Noble, the author, says in part

> "The purpose of this paper is to . . emphasize that design should provide positive safety at the speeds of which vehicles are now capable or may be capable in the near future" Reference is made "to the enormous loss of life and property (by reason of accidents) on American streets and highways" and the need for studying improvements to meet "the increased speed of the present and future motor car"
>
> "No other engineering structure has such a death record as the highway The motor car designer has set a pace which the highway

designer has not anticipated properly New automobile models appear each year but new highways cannot be produced so quickly This imposes on the designer the task of producing a highway suitable for the car of ten or fifteen years in the future Today (1936) there is a large road mileage less than ten years old which is hopelessly obsolete so far as the safe and efficient operation of the present motor car is concerned The injuries and loss of life and property on American highways are issues squarely facing the highway engineer It is his duty to design the highways so that the traveling public is safeguarded"

The general assumption that, because passenger cars are built capable of attaining speeds of 90 to 100 m p h , that highways should be designed to meet these speeds, hardly accords with the statements of conservative manufacturers General Motors Corporation points out that the most important reason for having power what it is in modern cars, is the avoidance of strain on the mechanism by not using the full capacity By building in the ability to run at high speed, engineers make it practical to run at reasonable speed and get better performance, dependability and lower cost of maintenance

This, of course, does not prevent certain drivers from attempting to utilize the full power provided but it is certainly open to question whether public monies should be spent to encourage this recklessness The statement that the highway structure is responsible for a large toll of accidents is also not borne out by any known facts

John S Worley, Professor of Transportation Engineering of the University of Michigan, states in *Engineering News-Record* of December 10, 1936

"There is a crying demand for highways which will permit safe and rapid movement of motor vehicles over long distances"

Whether it is economically sound to meet this demand through the expenditure of public money at the general expense of the taxpayers, and so endanger existing investments in already established means of safe and rapid transportation over long distances, he does not say He does, however, echo a similar statement made by the Honorable Murray D Van Wagoner, State Highway Commissioner of Michigan, who, at the 21st Michigan Highway Conference in February, 1936, said

"The State Highway Department is faced with the almost unanimous desire on the part of motorists for greater speed We find this expression of speed in every phase of modern times, in business, industrial and social activities"

The State Highway Engineer of Oregon, in an article in *Engineering News-Record* of May 23, 1935, said

"Improvement in the design of automobiles has a tendency to make highways obsolete the practice (in Oregon) now is to design all trunk highways, except through mountains, for vehicular speeds of 75 to 100 miles . There is satisfaction that these standards of design have been justified by steadily increasing average speeds at which traffic is moving over major highways"

No mention is made, in this article, of traffic density or highway capacity as affected by speed, or speed as affected by traffic density There is also probably some economic relation between the original cost and annual cost of such highways and the costs of moving traffic, values of time saved, etc , to which no reference is made

In examining the standards of the Oregon State Highway Commission, courteously furnished by the State Highway Engineer, it is noted that the roads of that State are divided into seven classes, A to G, and that it is only for Class A roads that design standards have been established of 100 m p h in flat country, 75 m p h in rolling country, and 60 m p h in heavier topography and that the *"recommended safe speeds"* for roads so designed are, respectively, 65, 65 and 50 m p h The maximum curves are also given at 2°, 6° and 10°, respectively, for the three classes and it is noted that these Class A roads are all designed with roadbed widths of 46 ft It is further noted that on Class A roads the super-elevation of curves only gives "full compensation" at 70 m p h as a maximum, and where the critical (designed) speed is 70 m p h the super-elevation only gives full compensation at about 36 m p h

There is evidently here a real recognition of economic limitations which modifies the statement, taken alone, that these highways are designed for speeds of 100 m p h

In Proceedings of the American Society of Civil Engineers of December 1936, in a discussion of "The Modern Ex-Press Highway," Mr T T Wiley states that

> "The highways being built today are scarcely abreast of the car of today not to mention the car of the future The highway engineer must exercise his imagination to visualize the conditions that will exist in the years to come, and base his work upon the conviction that the best design is scarcely good enough"

The context indicates that "best design" means "design of the highest type of roads which the present state of the art makes possible" Here again there is no discussion or mention of the fact that the designer, or at least the highest road authority, has many varied needs to meet and must do the best he can with the money available, which in fairness to all interests may or may not permit such "best design" to be translated into actual construction of all new or improved roads

Rowland Rogers, writing in the New York *Times* of April 2, 1937, advocates that highway builders take a lesson from American railroad practice He says

> 'Our American railroads have a low-accident record, both day and night When they increased the volume of traffic and the speed of trains, they did not illuminate their rights-of-way but they did
>
> "1 Build adequate roadbeds
> "2 Provide adequate sight distance for the engineer
> "3 Separate (divide) travel lanes (space between tracks)
> "4 Eliminate sharp curves
> "5 Reduce steep grades
>
> "In principle and in safety practice, both the rail locomotive and the highway locomotive (automobile) moving from 30 to 60 miles an hour, deserve adequate roadbeds and safe rights-of-way both day and night"

Like many others he seems to ignore the very varied ability, intelligence (or lack thereof) and responsibility of the human element in the case of the automobile. One may wonder, especially in the case of single track lines what would happen if a hundred or two minds of very varying qualities were directing the operations of trains, deciding when to stop and start, at what speeds to travel, etc, or when they might violate the rules and get away with it The physical condition of the roadbed, track, signals and cars would in that case have little to do with safety

Mr Arnold H Vey, Traffic Engineer of the New Jersey State Highway Department, while recognizing the value of certain types of improvements as accident preventatives, notes the fallibility of the human element In a paper presented to the Highway Division of the Am Soc C E in January, 1937, he said

> "By applying known highway improvements such as controlled rights-of-way, physical separation or segregation of conflicting flows of traffic, adequate shoulders for stopping, pedestrian pathways, and modern lighting, it would be possible to reduce accidents by 75 per cent Such changes make it less possible for highway users to perform improper practices causing accidents Even these improvements, however, are not cure-alls it is necessary to cause drivers and pedestrians alike to acquire safe highway habits"

In this he does not discuss the increased risks due to higher speeds, and it seems dangerous to hold out the hope that accidents can be decreased by anywhere near 75 per cent by construction methods In a very able discussion of this general subject, however, in a paper entitled "Highway safety exemplified by properly designed and constructed highways," presented at the meeting of the American Association of State Highway Officials at San Francisco, Cal, in December, 1936, Mr R E Toms, Chief of the Division of Design of the United State Bureau of Roads, says

> "On the face of the record it would seem absurd to use the wealth of the nation in building so-called fool proof highways A much more logical approach to the problem would be to expend the proper amount of effort to keep the fools off the highways"

Dr Miller McClintock is reported by the New York *Herald Tribune* as saying, at the national planning conference at Detroit on June 1st, 1937, that "the American street and highway system must be scrapped and rebuilt unless the automobile is to become a malignant growth" This, he says, "will cost $57,000,000,000 but will pay for itself in 43 years"[3]

There can be little valid argument against the exercise of foresight or reasonable planning for the future but the observation of the present writer is that little has been accomplished in the United States by long range planning or planning on a large scale True, if our foresight were as good as our hindsight we might have avoided some of our present difficulties but the engineer has to be continually on guard to avoid fantastic gestures and limit himself to that degree of future planning which is within the range of reasonable possibilities of the then state of the art, and the dimensions of financial support

These examples and quotations of only a few of the statements made within the past two years are sufficient to indicate the general types of criticisms which have been made, and ideas which have been advanced either to indicate that we do not know how to design proper highways or do not do so. Let us look first at our demonstrated ability in design where the necessity exists and the funds are available

SOME EXAMPLES OF CONSTRUCTION TO MEET MODERN REQUIREMENTS

That American engineers, given the opportunity and money are quite capable of meeting the demands of modern highway traffic, is evidenced by the work they have done Much of this work also shows an appreciation of future requirements

The Westchester Parkway System was started nearly a quarter of a century ago, developing a 40-ft 4-lane highway with no (or very few) grade crossings with other highways and none with railroads, and protected from side encroachments or entrances Of course the later sections built have reflected improvements in design (location) of the roadway but the general scheme developed[4] a quarter of a centry ago was a broad gauge vision of the future and anticipated the present so-called "freeways," the Italian autostradi and the German super-highways (autobahen)

The New Jersey State Highway, Route 25[5] from the Holland Tunnel to Elizabeth, a modern express highway 13 miles in length with a 50-ft roadway designed for and carrying very heavy traffic, through the congested areas of Jersey City, Newark and Elizabeth had no real precedents for its design, its avoidance of crossings at grade and developments of proper access roads without traffic crossings[6] Of course there was the precedent of the Westchester County Parkways in the matter of grade separations at other highway crossings, and the development of means of access and depar-

[3] See also *Engineering News-Record,* June 18, 1937, p 909

[4] Jay Downer, Chief Engineer

[5] Maj Wm G Sloan, State Highway Engineer (Originally Route No 1 Extension)

[6] Highways as Elements in Transportation, F Lavis, Trans Am Soc C E, Vol 95 (1931), p 1020

ture at these points, but this particular New Jersey highway, because of the nature of the traffic and the highly developed urban and manufacturing sections through which it passed, had to and did follow railway precedents, duly modified, rather than highway or parkway precedents both in its design and construction

Other examples of accomplished facts are the West Side elevated highway in New York City and its extension northerly to connect with the Westchester County Parkways The George Washington Bridge and its elaborate system of approach and connecting roadways as well as its general purpose as a by-pass route for traffic which formerly passed through New York City The complicated Triboro Bridge which is not merely a bridge but a connecting traffic link between New York City and the Westchester and Long Island Parkways and through highway routes to the north and east The 38th Street Tunnels in New York City and their carefully designed systems of approach highways between New Jersey and Long Island

There are also the Long Island Parkways, the Wacker Drive and Lake Shore or Outer Drive in Chicago, the East Boston Tunnel and its approach roads, the Worcester-Boston highway with its dividing strip between opposing lines of travel, built several years ago, and others

All these are examples of the highest types of the highway designer's art which will compare favorably with anything of the kind anywhere in the world It therefore cannot be said with fairness and reasonableness that American engineers are not fully capable of meeting the situation or that they fail to realize not only what the present requirements are but also what future requirements are likely to be in reasonably providing both for traffic capacity and its safe conduct

Engineers in this country also have been among the first to recognize the economic aspects of the problems of highway transportation, relation of construction costs and design to costs of delays, values of rise and fall, limitations of gradients and effect of these on operating costs of vehicles, effects of curvature, etc , as well as the effect of construction methods and types on maintenance and subsequent annual costs

Much, perhaps most, of this work, however, represents attempts to overcome traffic difficulties in congested traffic areas in the vicinity of large cities or metropolitan areas and each case has presented a special problem in itself These cases almost inevitably involve extremely heavy expenditures as, for example, the $40,000,000 which was the cost of the 13 miles of Route 25 New Jersey, from the Holland Tunnel to Elizabeth, which did not include the expensive connections of Routes 29 and 22

A cost of from $500,000 to $1,000,000 per traffic lane per mile, comparable to the cost per track mile of metropolitan rapid transit subway and elevated lines, may be thought of as a round figure for the cost of such construction This, again, may be compared with a cost of from $50,000 to $100,000 per mile as the cost of ordinary first class high type two-lane highways in not too difficult terrain in the open country and of course without the elimination of grade crossings

In reference to speed, however, it should be noted that because of the density and volume of traffic on these highways speeds of 35 to 40 m p h are about the maximum obtainable

OTHER GENERAL HIGHWAY REQUIREMENTS

While the need for these expensive constructions in congested areas of many of the large cities of the country is being acutely realized, there is also a demand for through express highway routes in the open country It is proposed that many of these routes should be four-lane highways with a separating strip between traffic in opposite directions, highways with grade crossings eliminated or permitted at only long intervals, and with a strip of land on either side which would guard against encroachments and prevent the erection of buildings abutting directly on the highway These are the so-called "freeways" to which reference has been made frequently during the past year or so

Such roads, designed and laid out to meet these requirements, need not, of course, be entirely completed at the beginning but it is advocated, and with a good deal of reason, that where the necessity is apparent, complete designs should be made in the first instance, and the necessary rights-of-way obtained

It has been suggested that there be a system of trunk lines of this character laid out on three or four routes crossing the United States from east to west and other routes from north to south

In New York the Superintendent of Public Works has proposed to the Legislature of that State the construction of a system of 927 miles of four-lane trunk highways (without the elimination of grade crossings with other highways), crossing the State in both directions, north and south and east and west, to be built at a cost of some $300,000,000 and the Merritt Parkway is actually under construction in Connecticut from the end of the Westchester County parkways in New York to Bridgeport, Conn

It is evident, therefore, that American engineers are also fully cognizant of the needs of this phase of the situation and the requirements of highways for through routes in the open country when money is made available

Incidentally, it may be noted that one short section of such a four-lane highway now proposed by the State of New York, with exceptionally heavy grading but having no bridges or structures of any importance, is estimated to cost around $300,000 per mile

Before leaving this phase of the subject it may perhaps be worth while to refer to what is probably the outstanding development of recent years in the construction of multiple lane highways, namely, the actual physical separation of lines of traffic moving in opposite directions

This, of course, has come about through the increase in travel on such highways and the fact that smooth pavements, good alignment and favorable gradients make faster driving possible It was not foreseen by highway builders of only a few years ago but was adopted wherever possible as soon as its need was apparent It is, however, hardly fair to the road builders of a few years ago to accuse them of negligence or lack of vision for not having foreseen this need Neither is it correct to say that highways which do not have the dividing strip are obsolete On these roads there must be more care in driving and it is not unreasonable to ask that there should be

DISTRIBUTION OF HIGHWAY FUNDS

Criticism is made of the design of these main line highways, both urban and interurban, and it is claimed that due consideration is not being given to the speed capacities of modern automobiles, that is, in providing long sight distances, and freedom from obstruction Roads of high character, however, have become necessary because of the need of accomodating large volumes of traffic, and maximum traffic capacity is not attained with vehicles moving at extremely high speeds

No doubt many of the proposals for super-highways, parkways, freeways, and express highways have merit and may be economically justified by savings in the costs of operation of the vehicles using them, but highway officials are compelled to have constantly in mind the requirements of all the people in the territories over which they have jurisdiction, and the need of making a fair distribution of such funds as may be available They must consider the farmer going to market as well as the pleasure seeker and fast driver

A notable example of adequate appreciation of this part of the problem is the recent report of the New York State Highway Survey Committee which first made a most comprehensive survey of the highway needs of the State and the present and probable future use of the highways, and then proposed the following program of constructing, reconstructing and rehabilitating the highway net·

(a) State Highway Reconstruction Program (5 yrs)	$138,000,000
(b) Construction to Complete the State System and including parkway construction outside cities (8 yrs)	80,000,000
(c) Arterial streets and by-passes outside New York City (5 yrs)	35,000,000
*(d) Construction to complete County Road System and reconstruction	124,000,000
*(e) Construction to complete Town Highway System	66,500,000
*(f) Arterial Highway System in New York City	60,000,000

* Time within which system will be completed not shown

A similar study was made and program proposed for a coordinated highway net by the State Highway Engineer of New Jersey in a most comprehensive report made to the State Highway Commission of that State in 1926 There are, of course, many other similarly comprehensive studies so that this phase of the problem has also received attention

GENERAL HIGHWAY REVENUES

It is not necessary in the present discussion to go into much detail in the matter of highway revenues and expenditures but a few general figures will indicate the dimensions of the problem

The total highway revenues for the whole of the United States for 1935 are stated by H Tucker in the New York *Times*, December 6, 1936, to be

For State Highway Systems	
Federal appropriations	$277,788,551
Traffic taxes *	950,456,000
For County Roads and City Streets	
Property Taxes (estimated)	1,000,000,000
	$2,228,244,551

* $135,710,214 diverted to other than highway purposes

In the whole of the United States the amount of taxes diverted to other than highway purposes is approximately 15 per cent

The sources of revenue for State, County and urban streets and highways vary but some general idea of what this variation is, is shown by the following figures presented by Thos H MacDonald, Chief of the United States Bureau of Public Roads, at the December, 1936, meeting of the Highway Research Board These figures are for nine States and are for the years 1930-1933

State Highways	
Motor vehicle taxes	$139,000,000
Federal aid	33,000,000
Property taxes	10,000,000
County Roads	
Motor vehicle taxes	$28,000,000
Property taxes	69,000,000
Urban Streets	
Motor vehicle taxes	16,000,000
Property taxes *	156,000,000
Miscellaneous	14,000,000

* $111,000,000 special assessments

It is estimated that there are 3,000,000 miles of roads in the United States of which only 14 per cent or 422,582 miles are included in State highway systems These latter are classified (New York Times, December 6, 1936) as follows

	Miles	Per Cent
High type	114,144	27
Intermediate	87 677	21
Low type	114.213	27
Unsurfaced	106,548	25
	422,582	

The problem of bringing all these roads up to a reasonable standard of usableness is a large one and would seem to require more attention than the building of speedways

THE GENERAL PROBLEM OF TRANSPORTATION

In considering the appropriation and application of highway funds, that is, public monies, to special needs, to encourage speed or even to facilitate movement, some consideration should be given to the transportation problem of the country The efficient maintenance and operation of our railway net is today, and so far as we can see into the future, an essential, even a vital need of our industrial and economic well being We should then give due consideration to the place of the highway, at any rate to that of trunk line highways and main travel routes as part of the general transportation system of our country So long as we have transportation needs of all kinds to be met, it is at least questionable how far we should go in duplicating facilities This may be increasingly important when we consider the ever increasing control of the railways by Federal authorities and the not improbable eventuality of governmental operation of the railways

John S Worley, Professor of Transportation Engineering of the University of Michigan, states in *Engineering News-Record* of December 10, 1936

> "Motor vehicles have been improved mechanically so as to make them capable of traveling over long distances at high rates of speed with a high factor of safety and economical cost of operation They are required (however) to be operated over highways which do not provide complete safety of operation and do not permit the full benefit of the very efficient vehicles we now have at our disposal"

In evaluating this statement there should be some thought of national economy as well as that of the motor vehicle and its users

Dr Harold G Moulton, President of Brookings Institute, quotes [7] Joseph B Eastman as stating "The time has surely come to deal with these matters with an eye to the general welfare of the community which, in the end, foots the bills" Reasonableness would seem to require that we consider the country as a whole (and the same arguments apply generally to almost any State as a whole) giving due consideration to at least two criteria to be applied to the expenditure of public funds for these main and trunk line highways.

A The relation of the highway net to other forms of transportation, and an equitable distribution of public funds based on this relation

B The relation of the trunk line highway net to secondary or other highways, and the equitable distribution of available funds to all the roads

It should be kept in mind that this present discussion refers to main routes and trunk line highways and does not take into consideration land access or property development roads which are mostly paid for by property assessment or local community taxes It must be remembered, however, that while these third and fourth class roads permit access to property they also permit owners of the properties to receive supplies, often previously hauled over main transportation routes, and to ship away from such properties goods or commodities produced there Such roads, therefore, evidently deserve economic consideration

COSTS OF HIGH SPEEDS

Prof R A Moyer calls attention in Proceedings Am Soc C E, May, 1937, p 940, to the very greatly increased costs

[7] *Railway Age,* November 14, 1936

of vehicle operation due to high speeds He says

> "Fuel costs at 70 m p.h are almost double the costs at 40 m p.h and there are indications that at 100 m p.h the cost is three times as much as at 40 m p h for the same car"

Costs of oil, tires, and repairs follow a similar trend and he calls attention to the fact that in automobile races, where speeds of 100 m p h or more are maintained for only 500 miles, engine breakdowns are frequent

As confirming this, it may be noted that in the Vanderbilt cup race held on July 5, 1937, on a special course at Westbury, L. I, only 12 out of 30 cars finished the 300 mile race, that the average speed of the winning car was 82 6 m p h and that of the slowest car which finished, 68 6 m p h

It has already been pointed out that conservative manufacturers consider that car design for speeds of 100 m p h is necessary, not that cars be driven at that speed but to provide the necessary factor of safety, for long service, at moderate speeds, without breakdowns, for economic operation and to keep the strain and tension on the driver within ordinary physical limits

The German Government recently (1937) made a test to try to determine the economic difference in operation over the ordinary highway between two cities about 100 miles apart and the new super-highways They found, among other things, that while an average speed of 74 4 m p h could be and was maintained on the super-highway as compared with an average speed of 44 4 m p h (maximum speed in both cases 80 m p h) on the ordinary highway, the gasoline consumption on the super-highway which had better surface, better alignment and required no stops was 5 5 gal as compared with 5 95 gal on the ordinary road

ACCIDENT RECORDS

That accidents on highways, due to the operation of motor vehicles, is a most serious problem has obtained general recognition and publicity in recent years Figures from different sources vary somewhat but those given below may be considered nearly enough correct to indicate the dimensions and characteristics of the problem

The National Safety Council reports that in 1935 there were 825,000 accidents to motor vehicles on highways, resulting in 37,000 fatalities, 105,000 permanently disabling injuries, and 1,180,000 temporary disabilities

The fatal accidents were classified (partly estimated) as follows

Due to collisions with	
Pedestrians	16,150
Other motor vehicles	9,650
Railroad trains	1,600
Street cars	250
Fixed objects	4,300
All other vehicles	550
	32,500
Non-collisions	4,500
	37,000

In the first quarter of 1937 there was a very decided increase, of about 25 per cent, in the motor vehicle accidents as compared with the first quarter of 1936

The classification as between types of vehicles, of fatal accidents, in 1936 is given as:

	Per cent
Passenger cars	77
Commercial cars (trucks)	18
Taxis, buses, motorcycles	5

This is some, very slight, indication of the effect of the skill of the driver on freedom from accident Comparing driving by day and night, the New York *Times* on November 22, 1936, said, "last year 21,480 people were killed in accidents between 6 00 P M and 6 00 A M, that is, at night, and 14,620 between

6 00 A M, and 6 00 P M, or during the day"

The National Safety Council in its 1936 report (p 30) states that "the open road should now be recognized as the location of today's most serious traffic safety problems Between 1924 and 1935 deaths from motor vehicle accidents in cities increased 27 per cent, whereas rural fatalities increased 150 per cent"

As long ago as March of 1924 the *Engineering News-Record*, commenting on the agitation at that time for the elimination of grade crossings of highways with railways, said in an editorial "the really dangerous place is straightaway pavement in an excellent condition of service" This is equally true today, over 13 years later, and is an interesting comment on the fact that improvements in alignment, gradients, surface, etc, do not necessarily insure safety

The effect of speed on fatalities is shown by the following tabulation of fatalities due to vehicular accidents at various speeds

0 to 20	m p h	1	accident	in	61	is	fatal
20 to 29	"	1	"	"	42	"	"
30 to 39	"	1	"	"	35	"	"
40 to 49	"	1	"	"	25	"	"
50 and over	"	1	"	"	11	"	"

It is stated that 9 out of 10 vehicles involved in fatal accidents were going "straight ahead" and in non-fatal accidents 8 out of 10 contemplated no turning movement

The report "Motor Vehicle Speeds on Connecticut Highways," Yale University, 1936 by Charles J Tilden, also points out the fact that not only is the severity of accidents increased with the increase in speed but the chances for such accidents also increase It was concluded from the observations made that high speed drivers have 45 per cent more accidents than do low speed drivers

To what extent is the highway structure itself responsible for these accidents?

Mr E C Lawson, Assistant Commissioner of Highways of the State of New York, in *Civil Engineering* of March, 1935, referred to the fairly well known fact that only about 5 per cent of all accidents are attributed to highway design, and that more accidents occur on straight roads than on curves

In the Proceedings of the Highway Research Board for 1935 Vol 15, p 430, it is stated by Arnold Vey, Traffic Engineer of the New Jersey State Highway Commission, that probably 85 per cent or more of all accidents on highways are chargeable to some improper action on the part of the driver or drivers involved

Mr Charles M Noble, however, states (Proceedings Am Soc C E September 1936) "many more accidents are chargeable to the highway itself than statistics indicate The failure to attribute accidents to faulty road design is due to several causes the principal of which is the subtlety of the problem—officials are not always trained to analyze the basic cause of accidents"

In Proceedings of the Am Soc C E for February, 1937, Mr J C Carpenter of the United States Bureau of Public Roads, compares the accident records on three roads in Texas, two old and one new, each of about the same length, the new road having excellent alignment, low degree curves, better sight distances, pavement 22 ft wide in excellent condition, fairly comparable traffic During the fiscal year ending August 1936 the accident records on those three roads were:

	Old Roads		New Road
Date constructed	1924	1922-1927	1934
Total accidents	15	45	64
Total killed	2	10	10
Total injured	26	48	49

Mr Carpenter points out also that good alignment and smooth pavement are likely to induce sleep and that 25 per cent of the accidents are caused by overtaking other automobiles, a difficulty which will not be remedied by dual roadways or other devices for "building safety into highways" On all roads speed is responsible for a large percentage of the accidents

Park Commissioner Robert Moses of New York, commenting in December, 1936, on accidents on the New York City and Long Island Parkways, is reported to have said, "Considering their traffic load the parkways are the safest arteries in the Metropolitan area" but he announced that "speed limits would have to be reduced and that police action is needed to cut down speeding" Such accidents as do occur, he said, "are the result of speed rather than bad lighting or improper construction"

The present writer pointed out (Proceedings Am Soc C E November 1936), "There should be taken into consideration from the standpoint of safety the effect on the driver of a monotonously even surface There is some evidence that this may tend to dull the senses and produce a state of at least semi-somnolence which does not develop in driving over roads where vigilance is obviously necessary"

This, of course, does not mean that we should go out of our way to build roads with crooked alignment and rough pavements but it does indicate that the reverse is not entirely warranted because of probable decrease in accidents on this account

The experience in England corresponds very closely to that of the United States In England and Wales, in the 5 years 1926 to 1932, inc, the "Failure of the Human Element" is stated to have caused about 84 per cent of all the accidents These were divided

	Per Cent
Drivers	32
Pedal cyclists	12
Pedestrians	39

The rest of the accidents were divided

	Per Cent
Vehicle defects	5.2
Road defects	5 6
Weather and misc	5 1

In a paper entitled "Road Design and Road Safety", presented to The Institution of Civil Engineers in December, 1936, F C Cook states that in Great Britain the total casualties on the road in the year 1935 were 6,502 killed and 221,726 injured.

This very interesting paper not only analyses the accidents and their causes but also the effect on accidents of remedial measures of design and construction, but the author concludes, among other things, that "The overwhelming majority of road accidents are due to the personal element and occur in circumstances for which the road *user* is primarily responsible" He further states that "out of this total of 6,289 accidents, 4,094 (two-thirds of the whole) occurred on the open road under conditions in which there was apparently ample visibility, whilst 1,883 were at junctions and 253 on bends and hills described as having 'bad sight lines'"

Mr Gilmore D Clarke, Landscape Architect to the Westchester County Park Commission and to the New York City Department of Parks, writing in Civil Engineering of April 1937 says

"It need hardly be stated that motorists are not capable of driving safely at that high rate of speed (100 m p h), and it is safe to predict that ordinary driving speeds probably never will exceed 50 m p.h At present we cannot afford to design roads for speeds much greater than this In the first place curves superelevated for even this speed are exceedingly dangerous in the winter months and, in the second, the capacity of a road upon which high speeds are permitted is materially reduced"

A very interesting report on certain aspects of accidents at grade crossings of highways with railways was made by Warren Henry, Assistant Chief Engineer of the Illinois Commerce Commission, at the Annual Highway Conference at the University of Michigan on February 17. 1937.

A detailed investigation in Illinois of crossings, crossing protection and the actions of drivers, showed that there, also, the human element was the most important factor

This analysis, which is lengthy and detailed, is particularly interesting as showing the general diffusion of accidents at all types of crossings Then taking one particular crossing, protected by mechanically operated lights, it was found that the warning was practically disregarded by some 49 per cent, or nearly half, of all the cars which reached the crossing after the warning signal first flashed Even up to 10 sec before the train arrived, about 15 per cent of the cars which reached the crossing proceeded to cross All this in spite of the fact that cars are required by Illinois State law to stop for red lights at railroad crossings as they are elsewhere

While automobile drivers persist in taking such chances with their own lives, with the lives of passengers in their cars, and with the lives of crew and passengers on the train, it is utterly futile to talk about "building safety into highways"

DESIGN TO AVOID ACCIDENTS

It seems to the writer, therefore, reasonable to conclude.

a That fatal accidents on highways are to a large extent due to the operation of private passenger cars

b That high speed of operation is an important factor in the causation of accidents, this especially, in view of the fact that collisions with pedestrians and other motor vehicles are the cause of about 70 per cent of all fatalities

It is, therefore, at least doubtful if the design of highways to permit or facilitate the driving of cars at high speed, or at least design which increases cost to obtain these purposes, is sound practice It is also doubtful if highways as they exist today are in themselves greatly responsible for accidents

It is true perhaps that with our present methods of reporting, all the causes of accidents are not made evident, and it is quite possible that defects in the road itself may have been contributory causes in some cases It is difficult for the present writer, however, to believe that defects, or lack of perfection of structure of the existing highways, are major or even important causes of highway accidents

Here reference should be made to the elimination of grade crossings of highways with railways Each of these crossings is a problem in itself, having as main factors to be considered, volumes of traffic on both routes, as well as sight distances, etc The elimination of many of these crossings is desirable but this, it seems to the writer, is an entirely different problem from that herein generally referred to which has to do with the layout, alignment, gradients, etc, of highways to permit or encourage high speed driving

It is also at least interesting to speculate whether or not nearer approach to perfection in the highway structure and its design to permit or facilitate the driving of cars at high speeds is desirable either from the standpoint of safety or that of sound practice, or warranted by any economic consideration

The writer considers it a fallacy to think, as some writers have stated, that highways designed for speeds of 100 m p h offer a factor of safety to drivers at moderate speeds (See previous note on Oregon Highways)

There is little doubt, where traffic demands make it necessary, highways of high class types have to be built The traffic requirements in these cases make it inevitable that on long sections of such routes the highest speeds of which modern cars are capable can be attained but it certainly is a matter for grave consideration as to how far we should go in projecting good alignment, low rates of gradient, and other improvements requiring large expenditures of public monies for heavy excavations, embankments and drainage structures, solely or largely to permit high speeds for a limited number of highway users and, so far as can now be judged, increasing the probability of accident The necessary curve elevations for the high speeds may also be a danger factor to cars driven at lower or normal speeds

As a matter of fact, where traffic demands are such that these expensive highways are required, the traffic density at times of maximum demand is such that speeds of 35 to 40 m p h are probably the most that can be attained Why then should such highways be designed so that at time of light traffic demand, which includes the night and early morning hours, speeds of 90 or 100 m p h can be attained with "safety"

What then are the conditions, if any, where design should take cognizance of speeds of 90 or 100 m p h or of speeds in excess of, say, 50 to 60 m p h which can probably be assumed to be the usual maximum, certainly the safe maximum today, for a large majority of drivers?

In considering this problem those who advocate designs based on higher speeds seem to have developed two main reasons

a That the difference in cost may be negligible and that the added factor of safety is worth this added cost

b That there is an ever increasing tendency to build automobiles capable of attaining the higher speeds, therefore, the highway engineer is neglecting his duty in not building to meet the demands which may be made by the drivers of such cars

DIFFERENCE IN COST AS AFFECTED BY SPEED

As to difference in cost, this probably will be affected by two major items

1 *Grading and Drainage* That is, the formation of the roadbed to permit longer sight distances due to easier horizontal curvature, longer, flatter vertical curves, and low rates of gradient

2 *Obstructions* Primarily, the elimination of crossings at grade with other highways or railways, providing wider shoulders, taking of additional rights-of-way to prevent encroachment, etc

SPEED CAPACITY OF VEHICLES

As to the second reason, the consideration of high speeds because automobiles are built which permit these speeds, and that the tendency of car design and construction is toward even greater speed possibilities and flexibility and ease of operation This gives no consideration to the fact that there are two factors which have to do with the speed at which automobiles are driven, namely, the driver and the car

It is the belief of the writer, although he has no definite proof of the fact, that a very large proportion (probably over 80 per cent) of the drivers of passenger automobiles (private cars) are incapable of driving with any degree of safety at speeds in excess of 40-50 m p h, very likely a good deal less than this Furthermore, it seems doubtful if ordinary drivers of private cars, who do not make driving their business, will, as a class, at any time in the near future, be capable of driving safely at speeds above 40-50 m p h , no matter what the road conditions may be or what improvements there may be in cars

It is claimed by Mr C M Noble, Proceedings Am Soc C E May, 1937, p 945, that the improvements in design of cars, steering, brakes, reduction in vibration, etc , have eradicated all sense of speed value, with complete lack of nerve tension and fatigue for the operator That this is to some extent true cannot be successfully contradicted, and yet it is the writer's observation, both as a driver himself (for nearly 30 years) and as an observer of professional drivers (chauffeurs) of private cars that it is only rarely that speeds of 60 m p h are exceeded even with modern cars

It may be admitted as true that accidents due to failure of the car mechanism, including tires, have been greatly reduced but they are far from being eliminated and it must, or at least should be, assumed that some element of failure may be present in any car It is of course also true that elements of failure are always present in even the best of drivers and to a greater degree in others less skillful

Taking these elements together, therefore, it seems inadvisable to permit or encourage driving at speeds over 50 or 60 m p h.

EFFECT OF SPEED ON HIGHWAY CAPACITY

Mr Chandler Davis has pointed out in Proceedings Am Soc C E March, 1937, p 592, that at speeds of 100 m p h 836 ft is required for an emergency stop of a car in first class condition with modern 4-wheel brakes (Longer distances for cars less well equipped or with drivers not fully alert mentally) This prescribes the spacing of cars and works out about 7 cars per mile if a speed of 100 m p h is to be maintained, an obviously absurd condition when applied to the expensive roads which may permit such speeds

He calls attention to the fact that the United States Army, after careful study, has established a spacing of 30 yards between trucks, moving in companies, at 40 m p h , that is, 54 trucks per mile of highways

In view of the fact, therefore, that money is not now available, and apparently for many years will not be available, to anywhere meet the general requirements for all highway users, it appears to be most unsound to build unnecessarily expensive highways of very limited traffic capacity to meet the demands of only a small proportion of all drivers

It is probably true that many drivers do not preserve the theoretically safe stopping distance between cars going at given (high) speeds, expecting to utilize some of the stopping distance of the car ahead This, however, is not safe practice and if we are considering design for safety it may well be considered sound practice to stay within the limits of the ordinary capacity of drivers which probably is below 50-60 m p h

In discussing "Design Principles and Traffic Speeds" Mr E C Lawton, in *Civil Engineering* for March, 1935, gives the following example of distances required for the safe passing of vehicles proceeding at speeds of 40 to 50 m p h

> "If a vehicle (No 1) is proceeding at a speed of 48 miles per h behind another vehicle (No 2), proceeding in the same direction at a speed of 40 miles per h while a third (No 3) is approaching in the opposite direction at a speed of 40 miles per h , a time interval of 40 sec and a sight distance of 2006 ft will be required for No 1 to pass No 2 "

He further points out that

> "With the present trend toward high speed for both private automobiles and trucks, the eight-mile difference in the rate of speed is a common occurrence on highways outside of cities and villages Under the conditions noted, it is evident that only 2½ operations of passing, theoretically involving 7½ vehicles, could occupy a given mile of highway at any one time

Of course, from a practical point of view it seldom happens that three identical operations are under way at the same moment on any one mile of public highway On the other hand, it will be clearly evident that where there are many operations involving the passing of vehicles on any given mile of two-lane pavement, the safe capacity of the roadway is indeed limited "

In the Proceedings of the Highway Research Board for 1935, Vol 15, pp 472-4, it is pionted out that the free moving speed of vehicles on a first class highway, with from 0 to 10 per cent of trucks, is nearly constant and equal to about 43 m p h

A report of tests made by the Transportation Committee of Yale University cooperating with the Connecticut State Highway Commission states that observations in Connecticut in 1933-34 showed average speeds in winter of about 43 5 m p h and in summer 39 to 40 m p h Mr M A Conner, Commissioner of Motor Vehicles, states that about two-thirds of the motorists, of their own volition, travel at a rate of less than 45 m p h

NEED OF HIGHWAY CAPACITY

As a matter of fact the great highway need of today is capacity to meet traffic demands Apparently also, and so far as can be judged by even the most far-sighted engineers and highway authorities, this is going to be the most important need for some years to come Furthermore, it is an accepted fact that maximum capacity is only achieved at comparatively low speeds, probably at speeds less than 30 m p h

The Superintendent of Public Works of the State of New York, in his annual report for the year 1936 (published May 1937), after enumerating the benefits of good roads says (p 5) "In spite of these indisputable assets which follow good roads, highway construction in practically every State in the Union has lagged behind the traffic demands The New York State Highway System is trailing present day requirements (of traffic capacity) the falling behind in highway construction has been especially noticeable during the past three years " (See note elsewhere in regard to gas tax diversion in New York)

SUPER-ELEVATION OF CURVES

There seem to be very decided differences of opinion in regard to the proper degree of super-elevation for curves, one theory being that it should permit the operation of cars with safety, i e , without side slip, in ordinary weather at the same rate of speed as is possible, or usually attained, on tangents on the same road

In easy country where flat long radius curves are the rule, this is probably practical but it is generally impractical in rolling or hilly country, and entirely so in even fairly rough country, to elevate the outer rim of sharp curves for the higher speeds

Curve elevation on railways is a compromise between that necessary for high speed passenger trains and that which will not unduly increase the resistance of slower and heavier freight trains This may mean a slow order or slow signal for some passenger trains It does not seem unreasonable to expect automobiles to obey proper signals or signs and slow down for sharp curves which may not permit speeds of even 50-60 m p h

There is a certain body of opinion which objects to variation in alignment, gradients and sight distances which require slowing down from whatever may be considered normal speeds on the greater part of the highway length The writer is, however, of the personal opinion that such objections are not valid

Transition Curves: There is some difference of opinion as to the value, if any, of transition curves, or spiralized curves on highways Inasmuch, however, as the modern passenger car is a very flexible easily controlled machine which occupies only 6 ft or so of width on a 10 or 11 ft lane, it seems to the writer to be an excess of refinement to introduce the transition curve in designs of highway alignment Even as a means of better fitting the alignment to topography it seems doubtful if spirals are likely to have much, if any, justification

The conditions which prevail on a railway where extremely heavy masses, with practically rigid connection between the wheel flange and rail, have to be changed in direction, do not prevail where the direction of the ordinary motor car weighing only a ton or two, moving on flexible rubber tires has to be changed Even with heavy trucks there is little comparison with the requirements for changing the direction of a heavy locomotive and train on a railway track

UNIFORMITY OF STRUCTURE TO PERMIT UNIFORM SPEED

Several writers have expressed the opinion that highways should be designed so that uniformity of speed may be maintained, that curves be so developed that drivers reaching them after having driven for some distance over straight or nearly straight stretches of road shall not have to materially decrease their speed

Mr E C Lawton, in *Civil Engineering,* March, 1935, after calling attention to the requirement of the New York State Highway Department for "sight graphs" to accompany maps of proposed new construction, makes the following comment

"When the motorist comes to a place where the sight distance is suddenly restricted, in a section of road where vision previously has been ample, the travel hazard has been increased Such unfavorable combinations are immediately disclosed by the use of the sight-distance graph Whenever special conditions unexpectedly reduce the vision at isolated locations, considerable time and money are expended to eliminate or improve them, particularly on main routes On the other hand, where a number of unfavorable situations rather close together are encountered in mountainous territory, the motorist is prepared by the gradual reduction in vision to operate his car in a safe and rational manner This feature of gradual reduction in vision is very important in highway design, particularly in rolling and mountainous country where the vision may be restricted for considerable distances"

Given all the conditions to which reference has previously been made, there seems to be no adequate reason for the assumption that road conditions should permit uniform speeds If curves are to be super-elevated such super-elevation should be gauged to meet the needs of the moderate or average driver, and proper signs should give drivers adequate notice of any change

Reference has been made to railroad safety as compared with highway safety but every locomotive driver has been trained from his first days on the railroad to watch for signs and signals He never knows when he may have to apply his emergency brakes, or at least to slow down Locomotive drivers are not only traditionally keen and alert, but are picked men whose physical condition is continuously checked It is little to ask of any driver of a high powered car, or for that matter of any car on the highway, to be ever on the alert for signs or signals requiring him to slow down or stop

Signs and warnings on highways should, of course, be conspicuously and clearly displayed, and there should be every endeavor made to have them uniform, but it seems to the writer that there is every reason for asking and expecting the drivers of motor vehicles to look out for, see and obey signs

The National Safety Council in its 1936 Report (p 26) says "The careful, skillful driver, however, rarely has an accident, even on a defective highway"

Here again it should be repeated that the fact that drivers can be warned and that warnings should be obeyed, is no reason for introducing unnecessary hazards or for not attempting, within reason, to remove hazards, but fair and reasonable judgment has to be exercised here as everywhere else

EFFECT OF SPEED ON DESIGN OF VARIOUS TYPES OF ROADS

Super-Highways or Arterial Highways This class of highways may be considered to include those built to relieve congestion in and in the vicinity of large cities It is doubtful if, from the very nature of such highways, speeds in excess of 40-45 m p h can be considered Such highways are designed for large volumes of traffic the general speed of which cannot properly exceed 40-45 m p h

There has been some criticism of the slow signs on the West Side elevated highway in New York where on good alignment normal speeds of 35-45 m p h are permitted but where at certain points where sharp reversed curves have been built signs require slowing down to 20-25 m p h So far as the writer knows, and has observed, these slow signs offer little difficulty and it seems doubtful if they decrease traffic capacity

Parkways and Freeways Highways of this class are also designed to accomodate large volumes of traffic and are only justified where such traffic exists They are located in more open country, generally in the vicinity of large cities, forming arteries of approach through more or less built up areas They are characterized by freedom from interruption of the flow of traffic by reason of the elimination of all or nearly all obstructions such as crossings of railways or other highways at grade, exits and entrances being only permitted in the direction of the traffic flow and at fairly long intervals

On these roads good alignment, moderate rates of gradient and consequently long or fairly long sight distances are desirable Being located in open or less densely built up country, costs of rights-of-way are usually not unduly increased by providing for reasonably good alignment Both alignment and gradients, however, have to be a compromise between topographical restrictions, costs of rights-of-way and costs of construction

They, therefore, in themselves may permit the attainment of high speeds It is doubtful, however, on account of the large volumes of traffic which they carry, and which justify the expenditures necessary to provide them, that speeds of over 40 to 45 m p h need be considered in their design, inasmuch as this is about the highest speed at which large volumes of traffic move

On account of the cost of these so-called "Freeways," the necessary acquisition of expensive rights-of-way, and the destruction of taxable values of lands so appropriated, it is doubtful if such highways will, for a good many years to come, form an appreciable part of the mileage of our highway system

The development of encroachments, however, especially gas stations, eating places, etc , with their attendant risks and reduction of highway capacity is one which is receiving considerable attention and in order to provide for such places, which are an accommodation to motorists, it has been suggested that definite setbacks be provided along important highways so that cars need not be parked or stopped on the pavement

Even with this provision, however, care will always be necessary to avoid collisions with stopping or starting cars and this is a precaution which drivers may well be expected to exercise

Traffic conditions and the number of accidents on the Long Island Motor Parkways have made it necessary not only to restrict speeds but to use a sufficiently large police force to enforce the restrictions As the result of several serious accidents in which speed had been a factor, these restrictive regulations were recently put into effect and in explaining them Commissioner Moses (New York *Times*, December 20, 1936) is reported as stating, "The parkways are the safest arteries in the Metropolitan area and such accidents as do occur are the result of speed rather than bad lighting or improper construction "

Main Trunk Highways These may be considered, for the purpose of this discussion, as either four-lane or two-lane highways in the open country

Modern design of such highways calls for high class pavements, lanes at least 10 ft wide (with some tendency to widths of 11 or 12 ft), separation of traffic moving in opposite directions, (at least on 4-lane roads), fairly good alignment and reasonable rates of gradient Most of such roads will probably avoid crossings of railways at grade, but will not usually avoid grade crossings with other highways, though the most important of these will be protected by signal lights

It must be borne in mind that the elimination of grade crossings with other highways not only involves very costly construction but lack of convenient access to main highways which may be a hardship to rural communities The desirability of eliminating crossings with railways has already been referred to

It is probable that in ordinary fairly easy country such roads might be expected to have curves with at least 1,000 ft radius, maximum gradients of 3 5 per cent to 5 0 per cent (the maximum to avoid slowing up of trucks with consequent reduction of traffic capacity) and sight distances of not less than 1,000 ft, sufficient under normal traction conditions to permit stoppage of a car with good brake equipment travelling at 100 m p h after the driver perceives the need of stopping

It is not, however, reasonable to expect the establishment of such conditions as this in many parts of the United States except in the prairie regions of the Mississippi valley As an example may be cited a 4-lane highway of the highest class, for which contracts were let by the State of New York early in 1937 The section referred to is a part of the main trunk highway on the west side of the Hudson River, and located near West Point about 70 miles north of New York City It is a 4-lane highway with division between opposing lines of travel, heavy concrete pavement, and is located in hilly to almost mountainous country, has fairly long sustained gradients of 7 per cent in which are short stretches of 7 6 per cent and 8 0 per cent, there is one curve of 410 feet radius on 5 48 per cent gradient, and minimum sight distances at the summit vertical curve of 425 ft

It is very evident of course that this highway will not permit, even if there were no other traffic, safe driving at 100 m p h and yet, so far as the details of location are concerned, and assuming that it is necessary to build the highway through this territory, there can be little criticism of the standards adopted, when the topographical conditions are taken into account According, however, to some of the citations made at the beginning of this paper this highway is obsolete before it is built

Where then, in view of all the statements previously cited that 100 m p h should be a normal standard of design, should this be applied?

As a matter of fact, the proposed network of 4-lane highways which it is proposed to build in the State of New York is said to be necessary to take care of a large volume of traffic

It is a pretty well established fact that fairly large volumes of traffic on good roads do not, and probably cannot, move at speeds over 40 to 45 m p h If traffic is very dense it tends to move at somewhat lesser speed, if not very crowded, speeds up to perhaps 55 m p h may be attained

It is evident that in easy country permitting long tangents, long radius curves and easy gradients, such roads in themselves permit the driving of cars at high speeds, but there is always the danger of the entry onto the road of vehicles from driveways to private property and from cross roads Omitting from consideration other traffic it would appear that speeds of over 90 to 100 m p h are inherently extremely dangerous and even 50 to 60 m p h involves the taking of chances on such highways As a matter of fact the Oregon State Highway Commission recommend 65 m p h as the "safe" driving speed on such roads, (designed for 100 m p h) presumably for drivers capable of "safe" driving at that speed

It would then appear that in more difficult country there can be no valid reason for adopting standards applicable to high speed for roads of any type, and even in easy country it is doubtful in any expenditure, certainly no very large expenditures, are warranted solely to permit high speeds

Secondary Roads In view of the foregoing it hardly seems necessary to discuss the design of secondary roads for high speeds The writer, however, happened recently to travel over a road in Virginia which may be considered typical of the so-called obsolete road The car was a heavy passenger car of the latest model of one of the more expensive makes driven by a very skillful professional driver

The section driven over, about 40 miles in length, runs through farming country of diversified topography There are some quite long stretches of tangent through gently rolling country, other stretches with a fair amount of curvature in heavier rolling country, and certain sections in fairly stiff hilly country

The pavement is of a light type of bituminous macadam about 18 to 20 ft wide, shoulders about 3 ft wide, built 10 to 20 years ago, and with a surface today which can be classified as only reasonably good It is laid practically on the surface of the ground with the minimum amount of grading so that the profile in places is quite irregular with short vertical curves and in places quite short sight distances It carries an average traffic of about 1,000 cars per day of which 20 per cent are trucks

It is possible for a reasonably good driver to attain speeds of 50 to 55 or even 60 m p h over about 60 per cent of the distance On the balance, it is necessary, for reasonably safe driving, and not considering other traffic, to slow down to say, 40 m p h and in a few places to 30-35 m p h

OBSOLESCENCE

According to some of the citations made at the beginning of this paper this road is evidently, in the opinion of the writers quoted, now obsolete and would be considered dangerous, but in the opinion of the present writer it is neither, and presents today a reasonably good highway for reasonable drivers for reasonable public use Unless the traffic should greatly increase it needs only reasonable maintenance of the pavement for some years and then perhaps gradual attention in improving certain sections over a period of years

We shall have in the United States for many years to come, for many more years than we can now look forward to, roads of all classes and of infinite degrees of usability Until the whole network is complete, therefore, the use of the term obsolete does not seem to be applicable

Obsolescence is generally considered by engineers as referring to that stage of development where an old structure or machine can be economically replaced by a new one by reason of the increased capacity or more efficient operation of the latter in producing sufficiently lower costs to warrant the replacement

In this sense, therefore, a highway can only be considered obsolete when the cost of its replacement is warranted by demands for additional traffic capacity or by such increase in operating efficiency or safety of the vehicles expected to use it as will warrant the expenditure In view of the fact that traffic in any considerable volume cannot move on any highway at speeds in excess of 40 to 50 m p h , its unsuitability for speeds in excess of this cannot properly be considered a factor of obsolescence

SAFETY

It seems entirely correct to state that all roads which are passable are "safe" for private cars in reasonably good condition, which are carefully driven by competent drivers Trucks on construction jobs move, even when loaded, over the roughest surfaces

What then is meant by "building safety into highways"?

For the purpose of this present discussion, this will be considered as referring to the design and construction of new roads or complete reconstruction of old ones It does not consider at this time incompetent or careless work or improper maintenance, either or both of which might be contributory causes of accidents on highways as elsewhere

On new work the writer understands "building safety into highways" means making such roads safe for such types of motor vehicles as may use them, driven at moderate rates of speed, by competent drivers Such drivers may expect to find on ordinary highways at certain places sharp bends or curves, may expect to find other cars entering or leaving the highway, or possibly cars parked or stopped, obstructing the main travelled way

All of these obstructions, or any of them, however, tend to reduce the traffic capacity of the highway and when or if greater capacity is needed some or all may need removal as an economic necessity Even in the original design the competent engineer will, of course, provide for such capacity as may be expected to be needed at some reasonable future date, not merely that existing at the moment These are matters of economics, however, rather than of safety

Consideration must be given also to the fact that under modern conditions, where the motor vehicle has come to be used almost universally as a means of transportation, and pays a fair proportion of taxes for road upkeep, motorists have come to expect better conditions than this, at least on main through routes They expect on such routes smooth pavements, fairly long sight distances and reasonable freedom from the obstruction of stopped vehicles, conditions which permit driving much of the distance on highways of the first class at a maximum speed of 50-55 m p h and sustained average speeds of, say, 30-40 m p h where there are not too many towns to be passed through

It seems reasonable to expect also that on important main through highways at crossing with other important highways, there will be a separation of grades, at others traffic lights, and at less important crossings warning signs Two factors may govern the decision as to which of these may be feasible and/or desirable, namely, availability of funds and economic justification (See Highways as Elements of Transportation Trans Am Soc C E Vol 95 (1931) p 1020) Here also safety is a further important consideration inasmuch as accident records show a fairly large proportion of accidents at intersections The cost of accidents and the economic value of prevention may be evaluated There should be also, of course, on all roads warning sign of sharp curves, steep gradients, changes in general character of the road or pavement, and any other features which require slowing up, watchfulness or more careful driving

GENERAL OBSERVATIONS

The writer wishes to repeat and perhaps emphasize the fact that he does not minimize in the slightest degree the need for continued and continuous study and adequate consideration of all the factors governing highway design, and nothing in this paper should be construed as absolving those responsible for the planning of new highways or the maintenance of old ones from making and keeping them safe for reasonably careful driving

He pointed out several years ago the need of coordinating design and construction with operation and that may be considered to include safety and ease of operation as well as costs of operation

Highway design can be divided into two main elements, (a) the location and (b) design of structures and pavement Location is used in the old railroad sense of the adaptation of alignment and gradients to topography Alignment and gradients affect safety of operation only in limited degree They may affect economy of operation

The densest passenger traffic on any railroad in the world, the New York subways and elevated lines, has been and is carried safely for years at fairly high speeds over curves of 135 ft. radius (90 ft on the old elevated lines)

Curvature and gradients do, however, cause increased expense of operation of railroad trains It is probable that they do also to motor vehicles on highways They certainly tend, in some cases of sharp curves and heavy gradients, to decrease highway capacity, that is, they decrease the maximum number of motor vehicles which can pass over the highway in a given time

Much may be done by the careful locating engineer to increase safety by careful adjustments of curvature and gradients, especially combinations of the two Here, as Wellington remarked in Economic Theory of Railroad Location, skill with the transit or at the drafting table may lessen the use of the steam shovel

Other factors of highway location and design, which may affect safety, are changes in characteristics from good alignment and easy gradients to heavy curvature and steep gradients, and perhaps more particularly abrupt changes of gradient without adequate vertical curves There should not only be adequate warnings of such changes in the character in the alignment but possibly also a transition section may be worked out by the use of the transit and a few brains to permit the motorist to accustom himself to the change

Abrupt changes in the character of the pavement should also be avoided in the interest of safety, either changes in the type of the pavement, transition from pavements with good traction to others which may be "slippery when wet", changes to narrow widths and so on Such changes in character must be inevitable on many roads for many years to come and, of course, here also appropriate signs should be erected and the motorist should be expected to watch for them The somewhat common custom of leaving narrow bridges at intervals along otherwise improved highways seems inexcusable except in the case of some long and very expensive structures.

It is, of course, to be expected that there will be improvements in design, since the science or art of highway building must progress as do almost all other matters with which we have to deal Those responsible for the building of new highways, especially roads with expensive pavements, should have an adequate realization of the fact that they are building structures for which a long life is to be expected and which are to be used by vehicles which are constantly and rapidly being improved Every reasonable provision should, therefore, be made for the continued usefulness of the structure for some time in the future

It is also fairly evident that having in mind the vast amounts being spent annually on our highways, the great need for additional traffic capacity which is pressing, and the needs of many kinds of users, that the highest type of engineering judgment, coupled with business sense and ability should be brought to bear in the laying out and design of the highway net as a whole

Giving all due consideration to the future, however, it hardly seems to the writer that there is any reason for assuming higher rates of speed than 50-60 m p h in the design of any road, and even lower speeds than this in difficult terrain It seems doubtful if the time is here, or likely to be here soon, where too much money should be spent in the removal of obstacles to fast driving unless in badly congested areas this can be shown to be economically justified An adequate uniform system of signs and markers should be sufficient to warn careful drivers of changes in character or of surface alignment, or other reasons for watchfulness.

It is doubtful if provision for higher speeds provides a factor of safety in the highway structure but, on the contrary, may tend to encourage unsafe driving, and increase the accident risk Both the National Safety Council in the United States and authorities in England note the increasing prevalence of accidents on the open straight stretches of highway where visibility is good

While it goes without the saying that continuous improvement may and very likely will be made in the design and construction of future highways of all classes, this does not mean in the true sense of the word that any highways are obsolete

The fact that it is possible to drive modern cars at 100 m p h does not mean that it is normally safe to drive them at this speed even on the best of highways, especially when the road is occupied by other vehicles Additional power over ordinary requirements permits flexibility and ease of driving but is a reserve force rather than one to be commonly used

Admitting that the condition of the highway structure may be a greater contributing cause to accidents than present forms of reports or the matter of their making indicate, there is still overwhelming evidence that the human element is the great and predominating factor in highway accidents

While it is undoubtedly the duty of the engineer to build and maintain highways so that they are safe for the normal traffic, normally conducted, which they are expected to carry, he cannot "build safety into highways" or "build accident proof highways" which will overcome the differences in drivers, their limited capacity and the general unreliability of many of them

Even if it might be admitted that design for speeds of 100 m p h is desirable on some highways, the additional cost of this (over design for 50-60 m p h) must be weighed against the needs of the whole highway net and the present urgent need for greater traffic capacity Provision for any special traffic requirement must be weighed and adjusted with the requirements of the whole highway net In this connection due consideration should be given to the added cost of operation of motor vehicles at high speeds and the small proportion of operators who may be properly allowed to drive at such speeds

One of the most important elements in highway construction is consideration of low annual cost per vehicle mile or per ton mile rather than provision for unduly high speed

While no one can deny the need of studying future requirements, it is probably not desirable to anticipate *unduly* future traffic needs in present day expenditures, first, because of the difficulty in obtaining money to meet even today's known traffic demands, second, because of the difficulty of forecasting future trends in highway traffic and, third, because of the added cost of interest charges for facilities or types of construction carried out now which may not be utilized for some time

It is in this last named phase of the situation in which mature judgment based on experience must be brought into play, evaluating the design of today in the light of the experience of the past and reasonable provision for the future

CONCLUSIONS

The prevalence of accidents on highways is a matter of grave concern not only to engineers and highway authoritie but also to the general public It may be admitted that defects in the highway structure, faults of design or construction or maintenance may be contributory to the heavy toll of lives, personal injuries, and damage causing heavy loss but there is overwhelming evidence to show that most of this is probably due to the human element, to improper or incompetent driving of the vehicles Excessive speed is one important contributory cause It seems probable that the large majority of drivers are not ordinarily capable of driving at speeds in excess of 30 to 50 m p h

The writer is of the opinion that where the location or design of highways for speeds of over 50 to 60 m p h involves added expenditure for the requirements of this additional speed such expenditures are not justified

The great need of today and very likely the great need for several years to come is increased traffic capacity Maximum capacity is only achieved at speeds considerably less than 50 m p h There should be recognition of the fact, however, that on uncongested highways speeds of 50-60 m p h may be permissible for competent drivers

It is possible that if and as funds become available, sufficient highway traffic capacity may be provided so that a freer movement of vehicles may be realized The expenditure of funds for this purpose, after reasonable provision has been made for highway users of all classes, may be economically justified by savings in time and possibly also in costs of operation if the speed at which cars can be most economically operated is more closely attained

DISCUSSION ON SPEED AND HIGHWAY DESIGN

Prof N W Dougherty, *University of Tennessee* There has been so much loose talk within the last few years regarding 100 m p h speeds and super-highways with suggestions that our present investment should be junked and that we start all over again, that I think it is high time for more thoughtful people to point out the inherent difficulty in such high speeds and the super-highway

I do not doubt the ability of the motor vehicle manufacturers to produce a vehicle which can travel from 80 to 100 miles an hour, and at a reasonable fuel cost I do not see much prospect, however, of making drivers over and changing their reaction time to fit with these high speeds With a reaction time of one-half second, a vehicle can leave the highway and be far into the adjacent field before the driver can make a move When Professor Moyer points to the fact that on a speedway built for the purpose, no more than 50 per cent of the selected racing drivers finish more than 500 miles, the fatality is too great

Improvements in alignment and grades are needed for the modest speeds of the average driver, but to suggest that all we have done should be junked is very misleading

Mr. Lavis has reviewed this attitude and I am in general agreement with his conclusions

PROF A DIEFENDORF, *University of Pittsburgh* A wise highway program is one which spends its income in such a manner that its construction and maintenance will serve the greatest number of citizens It does not consider political expediency or patronage as paramount to actual service Its operation should be similar to that of any well organized corporation, the stockholders, the motoring public, receiving dividends in service

The so-called super-highway (an undefinable term) which will permit motorists to travel thousands of miles at maximum motor car speeds of say 100 m p h can be built, but at prohibitive costs, and with use limited to ideal weather conditions and to high speed vehicles Can the motorist today safely travel at these speeds, even though the highway conditions are suited to high speeds? He can not Should the motorists be lulled into any let up in his mental alertness while driving at this speed say for one second, he has travelled 146 ft and if his car has left the highway in any of this distance the result is not hard to imagine I believe that a safe speed has been reached at 50 m p h and that our physical and mental limitations will keep it there

Pennsylvania is now facing a problem of this type in the new proposed South Penn Tunnel Highway This proposed highway runs from Irwin, Penna on Route Number thirty to Middlesex, Penna on Route eleven It is a new short route low gradient highway, a direct east-west road from Pittsburgh to Harrisburg It will have four 11-ft lanes separated by a 6-ft planting strip between the center lanes Maximum grades are to be 3 per cent, curves will be limited to one per mile with 6 deg maximum curvature There are to be no railroad or highway grade crossings No mention has been made concerning vehicular speeds but Mr Marhall, the chief engineer has been quoted as saying that there will be no speed limit, and the route will be privately policed

The tunnel highway will be located on the southern and western slopes of the hills reducing adverse conditions of snow, ice, and fog

The estimated cost of the highway is $50,000,000 It will be privately financed with no burden on the State road funds Through tolls collected at each of the nine tunnels en route the highways are to be self supporting and when paid for are to be turned back to the state as a free road

This project may in time answer some of the questions brought up in Mr Lavis's paper A high speed so called super-highway will be built without any unusual disturbance of a normal program and should the demand warrant other similar projects, a super-highway plan may be paid for by the people who demand it In our case Routes 22 and 30 will be open and free for those who care to use them

MR O L KIPP, *Minnesota Highway Department* In connection with this paper it would be interesting to be able to obtain information concerning drivers which is not being obtained at the present time Many do not drive sufficient mileage during the course of a year so that reaction is automatic or sub-conscious and consequently, to a certain extent at least, such occasional drivers must think out every action necessary It appears that the thinking time, added to the reaction time of the driver, might be a factor in determining safe speeds or safe speed limits As a result of reading this paper there appears to me to be one way in which we can help the future possibilities and that is by having ample right of way and keeping encroaching industries as far away as possible

AVERAGE SPEED AND TRAFFIC DENSITY AS A MEASURE OF ROAD CAPACITY

By B D Greenshields

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SYNOPSIS

Measurement of the utility of a highway is its ability to facilitate traffic The comparative utility of various road types may therefore be determined from an accurate measurement of the usual speeds of motor vehicles and the density of traffic on the roadways considered

The average speeds recorded in the investigation were 47.8 miles an hour for passenger cars on pavement, 43.3 miles an hour on treated-type A, 37 1 miles an hour on treated-type B, and 32 6 miles an hour on untreated loose gravel

For each of these values the corresponding average speeds for trucks or heavy commercial vehicles were 37 9 miles per hour on pavement, 32.5 on type A, 30 9 on type B, and 28.2 for loose gravel

From these average speeds the relative capacities were deduced to be for cars, gravel 1, Treated Surface A 1.33, Treated Surface B 1 15, pavement 1 47, for trucks, gravel 1, Surface A 1 30, Surface B 1.23, pavement 1 51

The relative values of time on the various surfaces are also discussed

This report grew out of a study of the speeds and spacings of vehicles on several types of road surfaces and the resultant traffic capacity of the roadways The surfaces included one high type pavement, two treated gravel roads and one of loose gravel

The investigation was conducted in 1933 as a project of the Engineering Research Department of Michigan University R S Swinton, Associate Professor of Engineering Mechanics at the University, was co-worker on the investigation

It is not the purpose of this paper to point to a definite conclusion, but rather to present a question worthy of review and discussion

The measure of the utility of a highway is its ability to facilitate traffic The comparative utility of various roadways may be determined from measurements of the usual speeds of motor vehicles and the density of traffic on the roadways considered

The results obtained from sufficient data to insure reliability gave an average speed on the pavement of 47 85 miles per hour as compared with 43 27 miles per hour for the same cars on one treated road, 37 10 for another group of cars on the other treated surface road, and 32 56 miles per hour on the loose untreated gravel

For each of these values there was a similar analysis for trucks or heavy commercial cars which gave corresponding speeds of 37 94, 32 50, 30 90, and 28 2 miles per hour respectively

The photographic method [1] was used in securing these data A summary of the results obtained on each of the several types of highways, together with a brief statement of the cost of construction and annual maintenance, is taken up in order This is followed by a correlation with principles derived in earlier studies and a more critical study of the reasonableness and limitations of the study

TRAFFIC STUDY ON PAVEMENT

The pavement studied is located about 19 miles west of Midland It carried the same traffic as the treated gravel section with which it is compared There are no intermediate cross roads carrying ap-

[1] Proceedings Highway Research Board, Vol 13, p 382

preciable traffic The average daily traffic in 1932 was 1384 cars and trucks

A roughometer constructed according to the specifications of the U S Bureau of Public Roads [2] and operated on a 1933 Plymouth car at a speed of 35 miles per hour gave an average roughness index of 1 25 to the mile on three round trips over the section

On August 30, from 11 A M to 5 30 P M , 341 cars passed over this road at an average speed of 47 85 miles per hour, while 64 trucks maintained an average speed of 37 94 miles per hour This should be compared with the rate of travel on the treated gravel section carrying the same traffic These are believed to be reliable averages because on a duplicate run made on the 26th of August, 146 cars passed the same observation station averaging 47 67 miles per hour

Traffic Study of the Treated Gravel Section of the Above Road (A) This section carried the same traffic as the pavement On August 30, 1933, from 10 30 A M until 5 30 P M , 361 cars travelled over the section observed at an average speed of 43 27 miles per hour. During the same time 49 trucks passed at an average of 32 50 miles per hour The reliability of these figures may be checked with the average speed of 41 5 miles per hour maintained by 94 cars at the station on August 22 The average speed of 20 trucks was 30 8 miles per hour on that date

The section chosen for each roadway surface was the best unhampered straight-away available The weather had been dry for several weeks A reading of 3 06 was obtained from 6 trials of the roughometer

Study of Traffic on Another Treated Surface (B) · The average daily traffic on this road was 411 vehicles per day The comparative roughness of this surface was 10 1 per mile One hundred and twenty-five cars were observed on August 24 to pass at an average speed of 37 10 miles per hour The vehicles composing the traffic were on the average older models than on the first two stations observed While the surface was smooth, its texture lacked firmness The slow speed indicates that the texture of the surface may be a more important factor in determining speed than roughness

[2] Public Roads, Vol 7, No 7, Sept 1926

TRAFFIC BEHAVIOR ON LOOSE-GRAVEL ROAD

Two sections on loose-gravel were studied The roughometer recording on the first section was 12 3 The average daily traffic was about 402 vehicles On September 1, 1933, between 9 A M and 5 P M , 105 cars passed at an average speed of 28 95 miles per hour Twenty trucks averaged 29 7 miles per hour

On the other section on August 26, 1933, 186 cars passed at an average speed of 33 65 miles per hour, while 16 trucks had an average speed of 27 01 miles per hour On August 27, 125 cars passed at an average speed of 33 99 miles per hour and 3 trucks at 21 3 miles per hour The average for 416 cars on the two locations was 32 56 miles per hour and for the 39 trucks 28 2 miles per hour

TRAFFIC CAPACITIES OF THE VARIOUS ROAD SURFACES

The maximum number of vehicles that may pass over a section of road in a given time is a function of both the speed and the average spacing maintained between vehicles Previous studies show that the average spacing is 21 feet plus 1.1 times the velocity in miles per hour. At such a spacing, the number of vehicles per hour that will pass a given point equals

$$\frac{5280\,V}{1\,1\,V+21}$$

This formula, however, shows only the capacity when the road is completely congested Under such a condition, the speed for all vehicles is the same and there is no minimum or maximum speed, but only the average speed which is maintained by all and determined by the slowest vehicle There is no particular speed which gives a maximum capacity, as was once believed, but speed and capacity increase together Hence the logical tendency to permit higher velocities

Highway capacity has been defined (page 218 Vol 10 Proceedings Highway Research Board) as the ability of a road to permit traffic to move at its normal rate of speed Congestion occurs when vehicles are retarded by those ahead Until such congestion occurs, the capacity of the roadway is a function of the speed of the vehicles The average speed of the traffic is then a measure of the relative capacity The relative capacities of the road surfaces studied are as in Table 1

TABLE 1

RELATIVE CAPACITIES OF ROAD SURFACES

Type of surface	Relative capacity	
	Cars	Trucks
Gravel	1	1
Treated surface (B)	1 15	1 23
Treated surface (A)	1 33	1 30
Pavement	1 47	1 51

THE COST OF TRAFFIC DELAYS

The slow speeds caused by a poor road surface result in a time loss to the vehicle and passengers By making a reasonable assumption of the value of time, the cost to traffic may be estimated In one case the value of a vehicle minute was estimated to be five cents (Report of a "Study of Highway Traffic and the Highway System of Cook County, Illinois, page 16, Bureau of Public Roads, 1925) Mr Fred Lavis estimates the value of time loss as 2 3 cents per minute, and for non-commercial vehicles as one cent per minute (Proceedings of the American Society of Civil Engineers, 1930, page 1369) S Johannesson, in Civil Engineering, March, 1933, page 149, estimates that the average value of a private passenger car minute is 78 cents, and of a truck, 1 49 cents

Mr Johannesson, in arriving at a value of vehicle time, makes use of the economic theory that the value of a thing is what people are able and willing to pay for it

Given the choice of two routes of travel, one shorter but more expensive, a certain percentage of drivers will take the shorter route and pay a toll Studies of toll bridges have verified this fact There is an annoyance factor that should be taken into account Of two alternative routes, the one offering unobstructed travel will attract the most traffic The annoyance factor can probably be measured by the variation in speed over the route In the one case, high average speed would be achieved by excessive speeds over part of the route

Nathan Cherniack, Special Traffic Consultant, Port of New York Authority, in a "Report of Traffic and Revenue Potentialities and Probable Traffic Characteristics of the Proposed Battery-Hamilton Avenue Vehicular Tunnel", March 15, 1937, gives the following monetary values as appearing to be those placed by motorists upon running time and avoidance of waiting

	Passenger cars	Trucks
a Savings in running time	1 1¢ per min	
b Avoidance of waiting time at ferries	5 3¢ per min	8 0¢ per min

If the most conservative of these values for time be applied to the various average speeds observed on the surfaces studied, the relative costs of time lost per 100 vehicles are as shown in Table 2 No time loss is assumed for the pavement as it shows the highest average speed

ECONOMICS

Example 1

Let it be required to estimate the value of the time that might be saved by changing the loose gravel to Surface Treatment (A)

Taking average traffic of 402 vehicles per day of which 15 per cent are trucks the daily difference in time cost between the two roads may be computed as follows

45 cents—9 4 cents = 35 6 cents = the excess cost for 100 passenger car miles on Loose Gravel over Treated Surface (A)

82 cents—29 5 cents = 52 5 cents = the excess cost for 100 truck miles on Loose Gravel or Treated Surface (A)

Of the daily traffic of 402 vehicles 85 per cent or 342 are passenger cars and 60 are trucks

For the existing traffic therefore the estimated daily saving in time value by changing from Loose Gravel to Treated Surface (A) would = $3\ 42 \times 35\ 6¢ + 0\ 6 \times 52\ 5¢ = 153¢ = \$1\ 53$ per mile The estimated annual saving would be $558 per mile

Example 2

Let it be required to set up a traffic load, based upon maintenance, capital, and time loss costs, at which a change from a treated gravel surface to pavement becomes desirable (see Table 4)

TABLE 2

VALUE OF TIME LOST

Type of surface	Value of time lost per mile per 100 passenger cars	Value of time lost per mile per 100 trucks
	Cents	*Cents*
Pavement	0 00	0 00
Treated surface (A)	9 4	29 5
Treated surface (B)	27 6	54 2
Loose gravel	45 0	82 0

TABLE 3

A STUDY OF TRAFFIC SPEED ON DIFFERENT ROAD SURFACES

	Treatment B	Ordinary Gravel	Treatment A	Pavement
Roughness*, units per mile	10 1	12 3	3 06	1 25
Number of cars	125	416	361	341
Average speed of cars, miles per hour	37 1	29	43	48
Number of trucks	48	39	49	64
Average speed of trucks, miles per hour	30 9	28	33	38
Comparative car capacity	1 15	1	1 33	1 47
Comparative truck capacity	1 23	1	1 30	1 51
Estimated value of time savings† per mile over ordinary gravel for 500 cars per day for 4 good-weather months	$106 14	0	$217 16	$274 50
For 70 trucks per day for 4 months	23 74	0	44 84	70 03

*Measured by a roughometer constructed according to specifications of the U S Bureau of Public Roads (Public Roads, Volume 7, September, 1926)

†Based on the estimates of S Johannesson, *Civil Engineering*, March, 1933, Page 149

TABLE 4

Item	Pavement	Treated Surface A
	Annual cost per mile	Daily cost per vehicle mile per day
Amortization of cost in 25 years at 5 per cent	$1,420	
Maintain	150	$1 00
Operation loss		0 46
	$1,570	$1 46

T—daily traffic at which a change becomes desirable $= \frac{1570}{1\ 46} =$ 1,075 vehicles per day

These costs are not offered as authentic and each solution should be made from data furnished by parties concerned Only the method is of interest The cost of the change is estimated at $20,000 per mile

SUMMARY

The ability of a road to facilitate traffic is increased by improving the surface or removing other physical hazards, such as sharp curves, to permit increased speeds The monetary value of the time saved becomes a factor in the justification of the improvement The number of vehicles that can pass over a roadway at a rate of speed determined by the excellence of the vehicle and the skill or desire of the driver unhindered by the road hazards is the true measure of highway utility

REPORT OF DEPARTMENT OF DESIGN

C N CONNER, *Chairman*

REPORT OF COMMITTEE ON SIGHT DISTANCES

JOSEPH BARNETT, *Chairman*

Senior Highway Design Engineer, U S Bureau of Public Roads

NEEDED RESEARCH FOR THE DETERMINATION OF SIGHT DISTANCES

SYNOPSIS

Two and three-lane highways should be designed with two minimum sight distances in mind, one to accommodate the passing of motor vehicles with safety and one to be considered a non-passing minimum It is desirable that the sight distance at every point on a highway should exceed the passing minimum This generally is not possible, but if drivers are to be encouraged to remain behind slower moving vehicles until it is safe to pass, safe passing sections must be encountered frequently

Sight distance adequate for passing with safety is the sum of three distances that traversed during perception time, that traversed while passing, and that traversed by an opposing vehicle Research is required to determine perception time, or the time elapsed from the instant the road opens up to the view of the driver to the time he begins the operation of passing, when the opposing lane is free of traffic, spacing of vehicles before and after passing, the acceleration of the vehicle, and the time of passing Research also is needed to determine many factors which influence most drivers in arriving at a decision regarding the passing of slower moving vehicles and to gauge the ability of most drivers to properly utilize long sight distances

At intersections research is required for many factors similar to those related to sight distances on the road, and for other factors such as acceleration of vehicles from a stopped position, which are related only to sight distances at intersections

All factors must be related to the assumed design speed of the highway and all research should be undertaken with this end in mind

The ability to see is of the utmost importance in the safe and efficient operation of a highway transportation system Rolling stock on a railroad is confined to a known path by fixed track, yet block signal systems and well trained locomotive drivers are necessary for safe operation The path and speed of motor vehicles on the other hand are subject to the control of drivers whose training is elementary If safety is to be built into our highways, it is vitally necessary that the road be opened up to view for a sufficient distance to enable the driver to pass overtaken vehicles without hazard and to control the speed of the vehicle to avoid encountering unexpected obstacles in its path

This paper lists and discusses briefly the factors which enter into the calculations for sight distances required for safe operation on the highway with the object in view of determining wherein research and investigation are required to arrive at proper valuation of the factors involved

ASSUMED DESIGN SPEED

The assumed design speed of a highway is considered to be the maximum approximately uniform speed which probably will be adopted by the faster group of drivers but not, necessarily, by the small percentage of reckless ones

If a highway is designed and constructed with reasonable margins of safety for operation at the speed thus defined, a safe highway will be provided for all users except a very few who may be termed reckless and whose unreasonable demands we are not justified in satisfying

The scope of this paper does not permit entering into a detailed discussion of assumed design speed It is mentioned because it is an important fundamental of highway design and factors for the determination of sight distance must be related to it

NON-PASSING SIGHT DISTANCE

The length of highway visible to a driver at every point on it should be in excess of the distance required to bring the vehicle to a stop before reaching a stationary object in the same lane when traveling at the assumed design speed of the highway This distance may be termed the safe stopping distance The values for the factors entering into its determination should be chosen conservatively in order that drivers who normally drive faster than the assumed design speed and drivers who do so occasionally also may avoid encountering obstacles in the road

The safe stopping distance of a vehicle may be considered as the sum of the following three distances

Distance traversed during perception time

Distance traversed during brake reaction time

Braking distance

Perception Time Perception time is considered to be the time elapsed from the instant a stationary object in the same lane is visible to the instant the driver comes to the realization that the object is stationary and the brakes must be applied

Perception time probably varies considerably and is dependent upon the speed of the vehicle, the optical ability and natural reaction of the driver, the distance to the stationary object the instant it is visible, the character and visibility of the stationary object, the portion of the stationary object which first comes into view, the relative position and visibility of roadside objects, the condition of the atmosphere, the time of day, the interference and distraction of other vehicles, and the effects of opposing headlights

Perception time generally is discriminative in character, that is the realization that the object is stationary comes to the senses of the driver only by association with other objects For example it may take a few seconds for most drivers to come to the realization that another motor vehicle in the same lane is stationary It is only by subconscious association with fences, trees, or other stationary objects along the road that one realizes that the vehicle is stationary An apparent increase in the size of the vehicle tells the driver that he is approaching it but does not tell him that it is stationary Increase in size of vehicle is apparent only when a large portion of it is visible, by which time the observer will have advanced a considerable distance beyond the point where the vehicle first was visible Sometimes perception time is very short Persons alongside a stationary motor vehicle convey to the driver almost instantly that the vehicle is stationary, but the top of a vehicle is visible some time before a person changing a tire, for example, comes into view Perception time for objects other than motor vehicles may be appreciably different than that for a motor vehicle

Perception time may be different for the two general cases of an object becoming visible around a bend or over a crest

Perception time, which may be a few seconds, has an appreciable effect on non-passing sight distance, each second requiring a distance in feet equal to nearly one and one-half times the assumed design speed in miles per hour Research on which to base values for use in design is needed

Sometimes objects moving slowly in the same direction as the vehicle present considerable hazard If the vehicle can be stopped before reaching a stationary object it can be stopped more easily before reaching a moving object, but the moving object may deceive the driver into a false sense of security so that he does not apply his brakes as quickly Perception time for moving objects may be different than that for stationary objects

Brake Reaction Time Brake reaction time is considered to be the time elapsed from the instant the driver realizes that an object in the same lane is stationary and the brakes must be applied, to the instant the brakes are effectively applied

Considerable research has been done on brake reaction time The average generally is found to be between ½ and ¾ of a second Many drivers react in less than half a second, many require more than a second, and many whose average brake reaction time is ½ sec occasionally require a full second or more

The variation in values found is too small to affect non-passing minimum sight distance appreciably The difference between ¾ of a sec, which is greater than most drivers require, and 1 sec, which will cover nearly all drivers, is but 22 ft for an assumed speed of 60 miles per hour No additional research appears to be justified

Braking Distance Braking distance is considered to be the distance traversed by a motor vehicle from the point where the brakes are effectively applied to the point where it is brought to a stop

There has been considerable research on braking distance The results indicate that the conditions under which the tests are made have an important bearing on the braking distance Important factors affecting braking distance are effectiveness of brakes, condition of tires, and character and condition of road surface Climatic conditions and the presence of moisture, mud, and ice also affect the results appreciably

The braking distance of a vehicle on a level highway may be determined by the use of the formula

$$d = \frac{v^2}{2fg}$$

in which d is the braking distance in feet, v the velocity of the vehicle in feet per second when the brakes are applied, f the coefficient of friction between tires and roadway, and g the acceleration of gravity The friction force is assumed uniform throughout the period of deceleration The friction force may not be uniform but the adoption of a uniform coefficient of friction leads to little error in view of the fact that most tests are made by measuring the distance required to brake a vehicle and then using the formula assuming uniform friction to determine the coefficient of friction

Tests on clean wet pavements result in friction coefficients at impending skid varying between 0 4 and 0 8 The National Bureau of Standards made some tests to determine the comfortable rate of deceleration and arrived at 16 1 ft per sec per sec, which is equivalent to a coefficient of 0 5 The Motor Vehicle Department of the State of New Jersey assumes a comfortable rate of deceleration of 17 4 ft per sec per sec equivalent to a coefficient of 0 54

For the purpose of determining non-passing minimum sight distance a value for the coefficient of friction which is some fraction of the average friction at impending skid and which is lower than that likely to be used by most vehicle operators should be chosen For an assumed speed of 60 miles per hour a variation in the value of f between 0 4 and 0 5 means 60 ft and between 0 5 and 0 6 means 40 ft in required sight distance There appears to be sufficient data on which to base a value for the coefficient of friction precise enough for all practical purposes Additional research therefore does not appear to be justified unless a change in highway and motor vehicle construction practice appears to have a radical effect on braking distance

When a highway is on a grade the braking distance may be determined by the use of the formula

$$d = \frac{V^2}{2g(F \pm \text{grade})}$$

in which "grade" is the percent of grade divided by 100 Light grades affect the result very little For an assumed design speed of 60 miles per hour and an assumed coefficient of friction of 0 4 a 3 percent grade, for example, affects the braking distance about 20 ft There is little justification therefore for extensive research for stopping distances on grades except that the determination of the effect, if any, of grade on the coefficient of friction may be desirable

SECTIONS SAFE FOR PASSING

Passing on highways in which two or more lanes are provided for traffic in each direction presents no unusual problem Crossing the centerline of a highway of this type, except to avoid impending collision, is highly reckless driving resorted to by so small a percentage of drivers that no provision need be made for passing with safety by utilizing opposing lanes It may be assumed therefore that passing will be done in a lane in which other vehicles will be traveling in the same direction and sight distance in excess of the non-passing minimum is sufficient for safe operation

Passing on two- and three-lane highways which constitute the bulk of our highway system must be accomplished on a lane which may be occupied by opposing traffic If passing is to be accomplished with safety the driver of the passing vehicle must see enough of the highway clear of opposing traffic so that if opposing traffic appears after he has started to pass he will have sufficient time to pass and return to the right lane without cutting off the passed vehicle and before meeting opposing traffic

The ideal two- or three-lane highway from the standpoint of sight distance is one in which the sight distance at every point is in excess of the minimum sight distance required for passing with safety Passing slower-moving vehicles on a highway of this character is retarded only by the presence of traffic on the passing lane, that is the opposing lane on a two-lane highway or the middle lane of a three-lane highway, within a distance required for safe passing

It is rarely possible or economically advisable, however, to construct two- and three-lane highways with sight distance at every point in excess of the minimum required for passing with safety, nor is it necessary to do so Drivers for the most part are careful and remain behind slower-moving vehicles at sections where sight distance is insufficient for passing with safety, provided their patience is not overtaxed Sections with sight distance sufficient for passing must be provided at frequent intervals if safe driving is to be encouraged

On the basis of the foregoing it is evident that two- and three-lane highways should be designed with two minimum sight distances in mind, one to accommodate the passing of motor vehicles with safety and one to be considered a non-passing minimum Where highways are designed with one minimum sight distance in mind sections with sight distances considerably greater than the minimum generally are constructed These sections, however, are the accidental result of topographic conditions They may or may not be long enough to permit passing with safety and the distance between them may or may not be short enough to encourage drivers to confine attempts to pass to these sections

Sight distance in excess of the non-passing minimum should be found at all points on a highway The research required for an accurate determination of non-passing minimum sight distance has been discussed

Sight distance required for passing is considered to be the sum of the following three distances

Distance traversed during perception time
Distance traversed while passing
Distance traversed by opposing vehicle

Perception Time Perception time in connection with sight distance required for passing is considered to be the time elapsed from the instant the road opens up to the view of the driver of the overtaking vehicle to the time he begins the operation of passing when the opposing lane is free of traffic Research is needed for an accurate determination of the perception time which will be sufficient for most drivers Each second represents in feet about 1½ times the assumed design speed in miles per hour

The Operation of Passing: Passing generally is accomplished by one of two methods, though combinations of both methods often are used

In one method the driver of the overtaking vehicle remains some distance behind the slow moving vehicle and when the road opens up to view he edges over towards the passing lane and accelerates at the same time When he reaches a point close to the slow moving vehicle he decides to pass if the road is clear By this time he is traveling at a higher speed at which speed he continues (or he may continue to accelerate) until he passes the slower moving vehicle and returns to the right lane If opposing traffic appears when he is about to pass he can brake and return to the right lane behind the slow moving vehicle

In the second method the driver of the overtaking vehicle slows down and remains behind the slow moving vehicle until the road opens up to view when he sizes up the situation (perception time) and if the opposing lane is clear of traffic decides to pass He passes the slower moving vehicle and returns to the right lane by accelerating

For purposes of design, minimum sight distance should be based upon conditions which require the longest sight distance for safe operation provided such conditions are encountered on the road frequently The method of passing therefore which results in the longer required sight distance should be assumed if it is used frequently Many drivers naturally use the second method of passing and the necessity for waiting until opposing traffic clears the passing lane often makes the second the only possible method of passing

Research is required for accurate determination of the spacing of vehicles immediately preceding and after completing the operation of passing and of the maximum acceleration, which will be less than that used by most vehicle operators These data may be supplemented or replaced by research leading to an accurate determination of the total time of passing

The time of passing may be developed as follows

t—time of passing in seconds
v—speed of passed vehicle in ft per sec.
a—acceleration of passing vehicle in ft per sec per sec
S—average spacing of vehicles immediately preceding and after completing the operation of passing

Distance traversed by passing vehicle $= vt + 2S$

Average speed of passing vehicle $= \frac{1}{2}(v + v + at)$

Distance traversed by passing vehicle $= \frac{1}{2}(v + v + at)t$

Equating

$$vt + 2S = vt + \tfrac{1}{2}at^2$$

from which

$$t = 2\sqrt{\frac{S}{a}}$$

It may be noted that for constant vehicle spacing and acceleration the time of passing is independent of the speed

In 1934 Dr H C Dickinson[1] made some tests in which the method of passing employed was the same as the second method described herein The time of passing was found to be very nearly six seconds and independent of the speed, which varied from 5 to 45 miles per hour The rear car, traveling at the same speed as the car ahead, started to accelerate from a position about 1½ sec in time behind the car ahead If this period of time was the time it took the rear vehicle to overhaul the slower vehicle, the spacing of vehicles was constant and explains the constant time of passing

The spacing of vehicles may not be the same for different speeds In 1933 Dr Bruce D Greenshields[2] used a photographic method for studying behavior of traffic from which he developed the following formula for the spacing of vehicles in a train when the speed is controlled by the first vehicle

$$S = 21 + 1.1V$$

in which S is the spacing in feet and V the speed of traffic in miles per hour The spacing of vehicles immediately preceding and upon completing the operation of passing may be found to be different from the above and different from each other but there is reason to suspect that they will vary with the speed

Dr Greenshields[3] later used the photographic data in an attempt to determine the time of passing and found it to be 10 or 11 sec for most drivers The large variation in results found by these and other investigators indicates the need for further research on this important factor in minimum sight distance for passing

Distance Traversed by Opposing Vehicle The distance traversed by an opposing vehicle which comes into view at the beginning of the operation of passing is the product of the total time of passing and the speed of the opposing vehicle This distance must be included in the minimum sight distance required for passing with safety The speed of opposing traffic must be assumed and generally is considered to be equal to the assumed design speed of the highway No research in connection with this factor appears to be necessary

ADDITIONAL FACTORS

The factors which enter into the determination of the minimum sight distance required to pass a single slower moving vehicle with safety are not the only ones which enter into the design of sections with sight distance adequate for passing on two- and three-lane highways of various assumed design speeds

A driver traveling at the assumed design speed of a highway naturally desires to pass slower moving vehicles This desire is reasonable and should be satisfied To what extent, however, should design be adjusted to satisfy this desire? If conditions are such that an appreciable percentage of drivers are tempted to

[1] H C Dickinson, "Distance Required to Overtake and Pass Cars," *Highway Research Abstracts* No 14—October 1934

[2] The Photographic Method of Studying Traffic Behavior, by Bruce D Greenshields, *Proceedings*, Highway Research Board Vol 13 (1933)

[3] Bruce D Greenshields, "Distance and Time Required to Overtake and Pass Vehicles," *Proceedings*, Highway Research Board, Vol 15 (1935)

pass, good design, from the standpoint of safety, calls for recognizing these conditions and providing adequate facilities for passing Research is needed for accurate determination of these conditions which may be described by the following questions

What is the maximum distance between the end of a safe passing section and the beginning of the next one which will not cause impatience and not tempt an appreciable percentage of drivers to pass at points unsafe for passing? This must be related to the probable speed of overtaken vehicles because the time of traversing this distance varies inversely as the speed at which the vehicles will be required to travel It also should be related to traffic density because of the possibility of the passing lane being occupied by opposing traffic when a safe passing section is reached

What is the minimum difference in speed which will tempt an appreciable percentage of drivers to pass? The greater the difference in speed the shorter is the sight distance required for safe passing

What minimum number of vehicles in a train will discourage passing? The sight distance required for passing with safety increases appreciably with each increase in the number of vehicles in the train This factor should be related to assumed design speed and to the probable speed of the train of vehicles

What is the probable spacing of vehicles in a train? This factor also should be related to the probable speed of the train of vehicles

SIGHT DISTANCE AT INTERSECTIONS

Some of the factors used in the determination of sight distance at intersections are not appreciably different from those used for sight distance along highways Others apply only to sight distance at intersections Research is needed for accurate determination of some of these factors The development of the theory of sight distance at intersections is worthy of a separate paper, but some of the factors which appear to require research are listed

Perception time at intersections is different in some respects from perception time along a highway The driver approaching an intersection must glance diagonally across two triangular areas, one to the left and the other to the right, to observe traffic in both directions on the intersecting highway and arrive at a decision regarding his own actions If he is on a non-preference road signed for stopping no problem presents itself until after he has stopped If it is not desirable to subordinate traffic on one road to that on the other the driver approaching an intersection must decide whether to slow down and at what rate or whether to come to a stop His decision must be based on the traffic he observes on the intersecting highway Research on the period of time required by most drivers to reach this decision is needed Angles of intersection other than right angles may affect the results

When a driver is stopped at an intersection he should see enough of the intersecting highway to be able to cross without being endangered by a vehicle that appears after he has started The visible length of intersecting road in each direction should exceed the product of the assumed design speed of the intersecting road and the time it takes the stopped vehicle to cross To determine the time which should be assumed for a vehicle to cross a road research is needed on perception time of the driver stopped at an intersection, the time it takes to start the vehicle, and the rate of vehicle acceleration from a stopped position, due recognition being given to the fact that gears must be shifted These factors probably would have to be evaluated separately for passenger vehicles and for trucks

OPTICAL ABILITY OF THE DRIVER

The research listed thus far is needed to evaluate factors which enter into the determination of sight distances required for safe and efficient highway operation In some cases the resulting sight distances are of little practical value because of the inability of many drivers to take advantage of them If, for example, it is found that on a two-lane highway on which the assumed design speed is very high the sight distance required for passing is very great it may be inadvisable to expend extra sums for providing such sections Some drivers may not be able to see that far and many may not be capable of correct judgment where such distances are involved It may indicate that the assumed design speed for a two-lane highway is limited

Research is needed to determine the maximum distances which most drivers can see and react for various purposes and under varying conditions both day and night Research of this character may not be needed as urgently as that which may be used as a basis for determining minimum sight distances There is no danger in providing unnecessarily long sections of highway open to the view of the driver

IMMEDIATE APPLICATION

The adoption of correct principles regarding sight distance in the design of highways need not be delayed because of the lack of data outlined herein Values for the various factors considered reasonable or on the safe side may be assumed and used until such time as research develops more accurate values.

For non-passing minimum sight distance two seconds for perception time, one second for brake reaction time, and 0 4 for the uniform coefficient of friction may be considered reasonable values They result in non-passing minimum sight distances equal in feet to about ten times the assumed design speed in miles per hour The variation is not uniform being greater at higher speeds and less at lower speeds For four-lane and divided highways a greater margin of safety may be advisable This may be secured by assuming a speed 10 miles per hour greater than the assumed design speed of the highway for sight distance purposes

In the design of two- and three-lane highways, sections safe for passing should be given conscious thought and not come about only as accidental results of fitting the highway to the topography A distance of one to two miles between the end of one and the beginning of another safe passing section may be assumed as a desirable maximum for two-lane highways, shorter distances should be assumed for three-lane highways, and advantage should be taken of every favorable location to effect the construction of frequent safe passing sections

For passing minimum sight distance it may be assumed that vehicles traveling at an assumed number of miles per hour less than the assumed design speed will be passed in the face of opposing traffic traveling at the assumed design speed If this speed difference is assumed to be 10 miles per hour, perception time is assumed to be 3 sec , and time of passing is assumed to be 9 sec the required sight distance for an assumed speed of 40 miles per hour is about 1,000 ft and for 60 miles per hour about 1,700 ft

DISCUSSION ON SIGHT DISTANCE

Dr B D Greenshields, *College of the City of New York*· Mr Barnett pointed out that the safe distance required for passing was made up of three items, the distance traveled during the perception time, that traveled while passing, and that traveled by the opposing vehicle I think that there should be an added distance for a safety factor

Perception time and reaction time might be combined Usually in taking brake reaction time we flash a light and then note the lapse of time from the flashing of the light until the operator puts his foot on the brake This I believe is the common conception of reaction time and includes the time of perception This combining would not change the character of the formula

In regard to the spacings between vehicles on the opposite lane, I think it has been found that the distribution of the time intervals between the appearances of vehicles on an unobstructed highway, follows the normal probability curve These intervals, varying from a few seconds to a maximum depending upon the density of traffic, are the opportunities one has for passing As Mr Barnett stated, if one assumes an average difference of speed the time for passing becomes a constant Passing is possible only when this much time is available from traffic on the opposing lane The average or minimum spacing has little significance in the study It is rather the variation from the average which offers sufficient time for passing

I agree with the finding that about 1,000 ft is the distance required for passing for average speeds This is the same result which I obtained from observations made in Ohio and reported at a previous meeting of the Highway Research Board The assumption was made in the analysis that all slower cars were passed by the faster traveling ones and that the passings were made only as sufficient time became available on the opposite lane. It was assumed that the drivers selected their own safety factors The results would be safe passing distances rather than minimum

Dr A R Lauer, *Iowa State College* I agree with the general presentation by Mr Barnett and also the comments made by Dr. Greenshields However, for the sake of clearness, I would like to point out the difference between *"sense"* and *"judgment"* Ordinarily in thinking of the term "sense", we mean in the case of vision, "Do you see a thing or not?" Perception or interpretation of markings, etc , on the roadway is somewhat another area It is merely an interpretation and does not take three seconds for realization There is the third category of which we still have to take account and which is most important of all That is judgment When you see an object in a store window, or on the road and after you decide what it is, then, "Do you want it or don't you want it? "Can you pass it or can you not pass it?" These decisions are matters of judgment It is a process more complex than "sensation" or "perception " The amount of time required for accurate judgment may run into minutes instead of seconds

The lack of good judgment partly accounts for the fact that younger drivers have more than their share of accidents It accounts for the fact that one of low intelligence is likely to have more accidents in passing It partly accounts for the fact that alcohol seems to cause many accidents It is often not a question of, "Do they see it?" but a question of, "Do they see it and know what to do?" I fully believe the time has come when high speed on roadways, such as the proposed Pennsylvania super-highway or the high-speed roadways in Germany, should be limited Who can perform safely at those high rates of speed? The fact seems obvious that we need to keep the public out of the air so far as piloting is concerned We don't allow any one to fly a plane until he is well qualified I think the application of scientific principles to the situation at hand should be, first, define our terms carefully, and second, determine the safety factor required to satisfy the needs of any emergency and, as Dr Greenshields said, make it *sufficiently adequate* I do not see any reason for limited non-passing areas The minimum passing distance, as Dr Greenshields has suggested, should be established and maintained Otherwise someone may try to pass at any point and there are short places where you cannot see far enough ahead on existing highways, which is the reason for many accidents Why not plan the highways so the poorest driver will have no excuse for his accident on grounds of inadequate sight distance? Give them enough room to stage a self-debate on the advisability of passing and have plenty of time to actually do it The maximum passing distance will be short enough at high speeds

MR BARNETT It appears to be desirable to separate perception time and brake reaction time As Dr Greenshields pointed out, brake reaction time is direct On the road considerable time may elapse from the instant certain conditions develop to the beginning of brake reaction time

The report of the Massachusetts Highway Accident Survey, made in 1934 under the direction of the Massachusetts Institute of Technology, records the results of testing some drivers under varying conditions with both laboratory and road tests Among the latter were tests in which the stop light was covered and the rear driver was instructed to brake as soon as he noticed the forward vehicle slowing down I believe the contact between cars was made by radio The conclusion drawn was that "a driver who showed a reaction time of 0 2 to 0 3 sec under laboratory conditions might require 1 5 sec to apply his brakes under normal road conditions, if the stop light of the car ahead was out of order, after the driver ahead had actually applied his brakes" In other words their reaction time on the road, despite the fact that distraction was eliminated from the tests, was about seven times what it was in the laboratory

Reference has been made to a factor of safety A factor of safety may be applied in many ways It can be applied by assuming critical values for all factors and increasing the resulting sight distances It may be applied by assuming values below the critical for all factors such as a friction factor of 0 4 when tests on clean wet pavements indicate impending skid factors of 0 7 or 0 8

The concept of two minimum sight distances for an assumed design speed should be given consideration particularly when improving conditions on existing roads There seems to be little advantage in the practice of increasing minimum sight distances on existing two- or three-lane highways or increasing standards for minimum sight distances for new highways unless such increases provide the minimum sight distances sufficient for passing with safety This is particularly true on high speed highways on which minimum safe passing sight distances are much greater than the non-passing minimum sight distances Increasing sight distances by small amounts in excess of the non-passing minimums generally does not provide sight distances adequate for passing with safety and may make the highways more hazardous by encouraging vehicle operators to attempt to pass Few vehicle operators know or sense the distance required to pass a vehicle It seems to be more advisable to concentrate expenditures on limited sections of the highway where topographic conditions make such changes feasible and improve these sections to secure sight distances adequate for passing with safety

REPORT OF COMMITTEE ON ANTISKID PROPERTIES OF ROAD SURFACES

George E Martin *Chairman*

Consulting Engineer, The Barrett Company

APPARATUS FOR DETERMINING SKID RESISTANCE OF PAVEMENTS

SYNOPSIS

Two general types of apparatus have been used in the measurement of the slipperiness of pavements—trailers and single wheels attached at an angle to the propelling vehicle Description of types of apparatus includes that used by Moyer at Iowa, Stinson and Roberts in Ohio, the Ministry of Transport in England, and Boutteville, Bouly and the Syndicat S.F E R B in France Danger spots in a road system can be readily detected by routine testing of highways for the determination of the coefficient of friction between the tires and road The trailer type of testing apparatus possesses several advantages for this work Either the longitudinal coefficient or the sideways force coefficient is a satisfactory determinant It is pointed out that tires used should be smooth and kept inflated to a uniform pressure, that tests should be run at the highest practical speed—certainly not less than 30 m p h and that the pavement should be wet Tests of surface types are not sufficient, each individual section of road requires investigation Although no tests have been run over the same piece of road by the various kinds of testing apparatus, a comparison of results indicates a fairly close check

Slippery pavements still continue to take their toll of the motorist Present practice on the part of highway officials is to wait until several accidents occur on a stretch of road and then do something about it Routine regular testing of the road surfaces for their skid-resistant properties would make it possible to correct dangerous conditions before life and limb are sacrificed in determining that they are dangerous The City of St Louis has taken the lead in work of this sort The St Louis officials are using an apparatus similar to that developed by Professor Moyer Other responsible highway officials could profitably follow the St Louis procedure and test their road surfaces for skid resistance

Two general methods of measurement of skid-resistance have been used One a laboratory, and the other a field method The laboratory method has the advantage that the various factors can be controlled, but it cannot be applied to existing road surfaces New road surfaces tested under laboratory conditions do not always give the same results as when tested in the field Laboratory tests of a pavement type may be quite accurate, but tests of various examples of a type as built may give greater variation in results than tests of different pavement types The real question is whether a particular stretch of road is dangerously slippery or not Laboratory tests of types are of value but do not necessarily give this information The laboratory method of investigation is valuable as a research tool, but cannot be depended upon as a source of information for locating dangerous road conditions

In operating a motor car on the road, three forms of skidding may be encountered They are—(1) skidding straight ahead with the wheels locked with the brakes, (2) skidding straight ahead as the brakes are applied and the wheels are at the point of locking and sliding, and (3) skidding sideways on a curve without the use of brakes In practice, most skidding accidents are a combination of (1) or (2) with (3) This sideways skid will often occur on a straightaway as well as a curve

Machines have been devised which will measure the coefficient of friction between the tires and the road surface under these three conditions However, (2) is very hard to control, and most investigators have confined their efforts to the measurement of (1) or (3) or both

Two investigations have been carried on in the United States, one by Moyer in Iowa, and one by Stinson and Roberts in Ohio Both were reported at the 1933 and 1934 meetings of the Highway Research Board Trailers were used in both investigations

Two general types of apparatus have been used for measurement of the slipperiness of pavements—trailers and single wheels attached at an angle to the propelling vehicle The trailer type of testing apparatus seems to have some advantages in that its action is similar to that of automobiles and automobile tires may be used on it The apparatus can be used to measure the direct longitudinal coefficient or the sideways force coefficient The towing vehicle may carry a water tank so that the road can be sprinkled at the time of the test

The single wheel attached to the propelling vehicle is generally used only for measuring the sideways force coefficient This apparatus has a longer record of use than any of the others since it was developed for the Ministry of Transport in England, where more work of this kind has been done than in any other part of the world

The sideways force coefficient would seem to be the most simple form of comparison for routine testing of road surfaces The difficulties of braking action are eliminated, and only three factors need to be recorded, namely speed, wheel load, and sideways force The coefficient is the direct relation between the wheel load and the sideways force

A brief description of the commonly used types of apparatus will now be given

APPARATUS USED BY MOYER, IOWA STATE COLLEGE

This apparatus has been fully described in Vol 13, Proceedings, Highway Research Board and in Bulletin No 120 of Iowa State College from which the following has been abstracted

"The trailer was so constructed that it could be used interchangeably for the three forms of skidding Provision was made to vary the total load on the trailer from 630 to 1,630 pounds However, as a result of a study of the effect of variations in the weight of the trailer on the coefficient of friction and because of the ease of operation of the trailer using a light load, a standard gross load of 830 pounds was adopted With this load, it was possible to run tests satisfactorily and safely at speeds from 3 to 40 miles per hour

"For sprinkling the surfaces and to provide the power necessary to run tests at high speeds, a Graham truck, 2½ ton capacity, was equipped to tow the trailer The surface was sprinkled directly in front of the test trailer during each test Whenever the occasion presented itself, tests were run during and following rains

"To measure the skidding forces in the line of travel, the trailer was connected directly behind the towing truck (Figs 1 and 3) The tongue of the trailer was supported in a rocker arm maintained in a vertical position during the tests to eliminate the possibility of transmitting the horizontal pulling force from the truck to the trailer in any way except through the dynamometer The trailer was equipped with self-energizing mechanical brakes which were operated manually by means of a long brake lever conveniently located near the observer's seat on the truck Provision was made for a quick and easy method of adjusting the brakes and no difficulty was experienced in locking the wheels However, when first running tests for impending skidding, it was found that uniform braking distribution could not be maintained between the two wheels because of the inequality in the braking force on each wheel and partly because of the difference in the line of motion of the towing truck The 15-degree angle was selected on the basis of angle variation tests in which it was found that the coefficient reached a maximum value at an angle of about 12 degrees and remained constant for angles of inclination up to 30 degrees, the

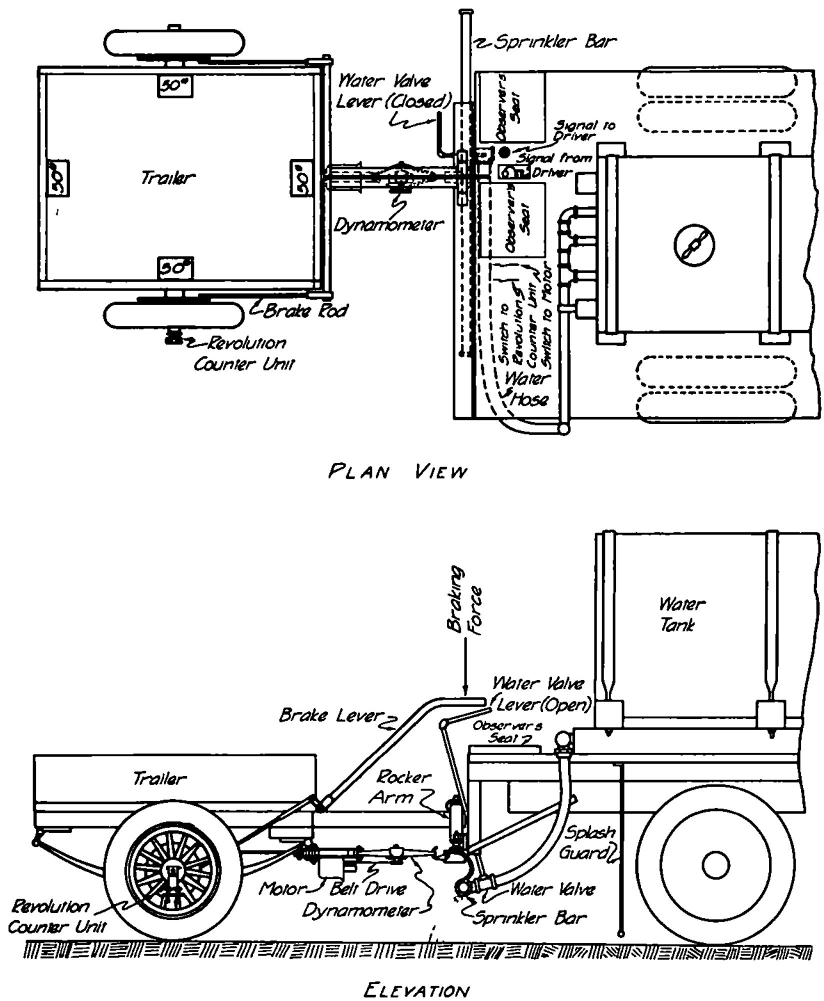

Figure 1 Arrangement of Test Equipment for Determining the Coefficient of Friction of Tires on Road Surfaces when Skidding Straight Ahead

the tire treads and road surface conditions To obviate this difficulty, the gears in the differential housing were cut square and a locking device inserted forming, in effect, a single axle

"In measuring the side skid forces, the trailer was connected to the towing truck in a position (Fig 2) such that the longitudinal axis of the trailer made an angle of about 15 degrees with maximum angle at which tests were run The integrating dynamometer was connected in line with the axle of the trailer and measured the force which caused the wheels to skid sideways As the towing truck moved forward, the trailer tended to swing into the direction of travel of the tow truck, thus simulating the action of a car skidding on a curve

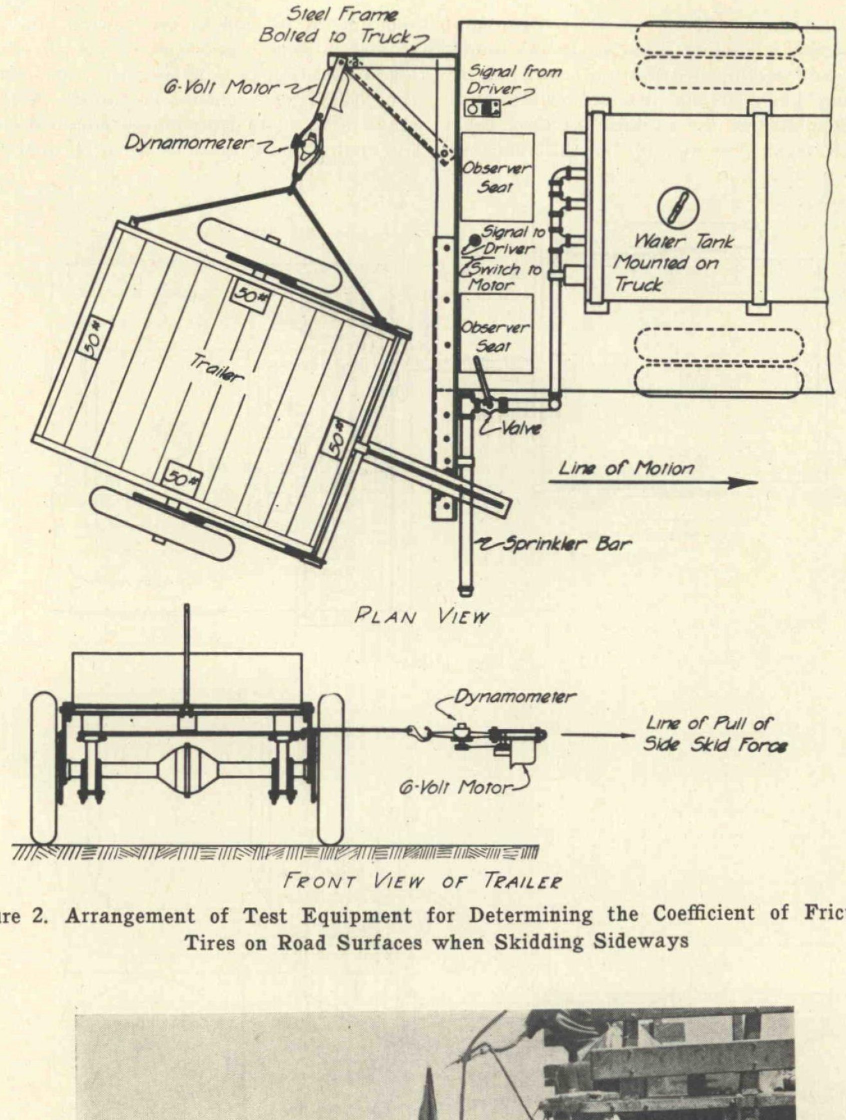

Figure 2. Arrangement of Test Equipment for Determining the Coefficient of Friction of Tires on Road Surfaces when Skidding Sideways

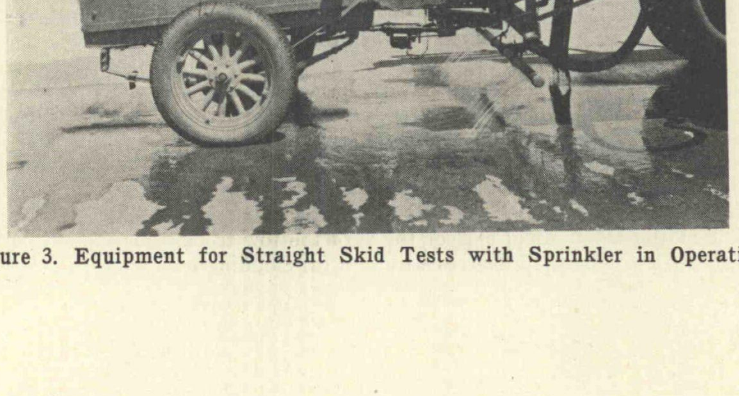

Figure 3. Equipment for Straight Skid Tests with Sprinkler in Operation

"The most important piece of equipment developed in this investigation was the integrating dynamometer (Fig. 4). Considerations governing its design were that: (1) The dynamometer should be simple and rugged in construction, capable of withstanding considerable abuse, and easy to attach to the trailer and tow car; (2) the human equation in reading or measuring the forces should be eliminated; (3) it should measure accurately to within 10 pounds forces ranging from 100 to 2,000 pounds; (4) the forces should be measured directly to eliminate the possibility of lag or the setting up of inertia forces in any part of the dynamometer; (5) it should be possible to calibrate the dynamometer under conditions of loading similar to that obtained during field tests; and (6) frequent field checks of the calibration should be easily made.

"In the search for a dynamometer meeting these requirements, preliminary testing was carried out with several dynamometers. The Kohlbusch dynamometer with maximum indicating hand measured only the maximum skidding force and this accurately only to the nearest 25 pounds. Calibrations of the Kohlbusch dynamometer spring with a Federal dial mounted in the frame indicated that an accuracy within 5 pounds could be obtained. Tests with a heavy duty integrating traction dynamometer developed by the Agricultural Engineering Department of Iowa State College indicated that there was considerable lag in the spiral spring and the levers of the recording and integrating mechanism. However, it was recognized that a dynamometer meeting all the requirements set forth could be made by mounting on the Kohlbusch dynamometer spring an integrating device of the same general type used in the traction dynamometer.

"The integrating device (Fig. 4) designed for this purpose consisted of a rotating fiber disk and two revolution counters. The fiber disk was mounted at a fixed distance from one side of the dynamometer spring. The inside revolution counters were geared to the disk and recorded its revolutions to the nearest tenth of a revolution. The disk was rotated at a constant speed by a motor drive. Another revolution counter, which served as the force-distance integrator, was fastened to the other side of the dynamometer spring and recorded the revolutions of a small steel wheel driven by friction on the fiber disk. When pull was applied to the dynamometer, the steel wheel moved toward the center of the disk, and turned with fewer revolutions in proportion to the number of revolutions made by the fiber disk. The quotient obtained by dividing the number of revolutions of the disk by the number of revolutions

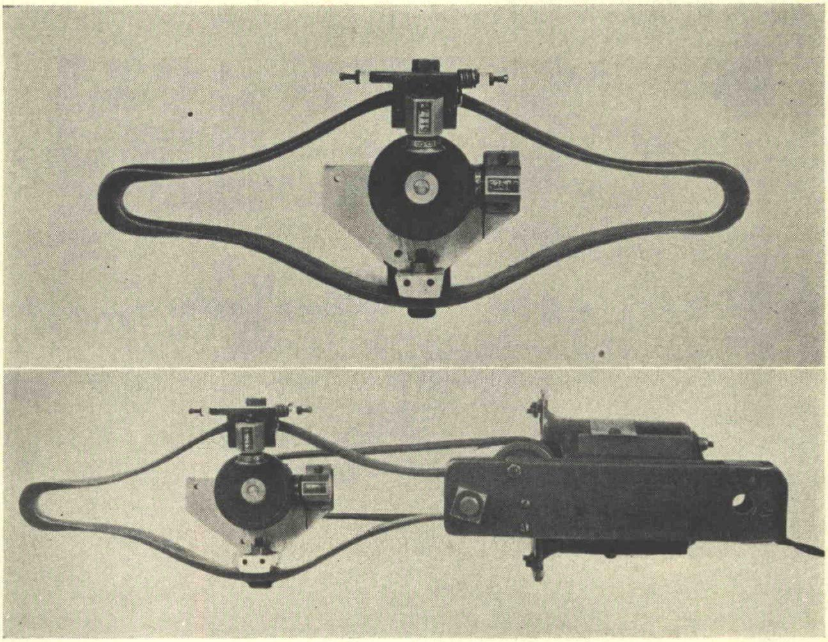

Figure 4. Integrating Dynamometer and Dynamometer with Motor Drive

of the steel wheel was a measure of the pull transmitted by the dynamometer.

"A useful field check on the calibration of the dynamometer was obtained by checking the quotient for zero pull after each series of tests and as frequently as six times a day. Slightly roughening the edges of the steel wheel kept it from slipping on the disk. The field calibration for zero load served as a means for detecting slippage at this point. Laboratory calibrations were made at least once every two weeks during the period when tests were run. The remarkable consistency of calibration and field results indicate the ruggedness and dependability of the dynamometer.

"Although it is desirable to know the maximum and minimum skidding forces, the most satisfactory basis for comparing the skidding characteristics of road surfaces is on the basis of the average coefficients of friction. In tests by Agg the vibrations set up in the trailer during the tests were generally responsible for fairly large variations in the magnitude of the skidding forces. The average pull measured by the integrating dynamometer eliminated, for all practical purposes, the effect of these vibrations."

APPARATUS USED BY STINSON AND ROBERTS, OHIO STATE UNIVERSITY

This apparatus has been fully described in Vol. 13, Proceedings of the Highway Research Board from which the following has been abstracted and brought up to date by Professor Stinson:

"The apparatus consisted of: a trailer (Figs. 5 and 6), a hydraulic dynamometer (Fig. 7), a recording and controlling mechanism (Fig. 8), and a towing car.

"The trailer was made from the rear end of an automobile chassis and carries a conventional load for the tires used. Only the left wheel of the trailer is equipped with a brake. This eliminates the need of equalization and permits the determination of both rolling and sliding coefficients of friction in the line of travel. The draw bar is attached in line with

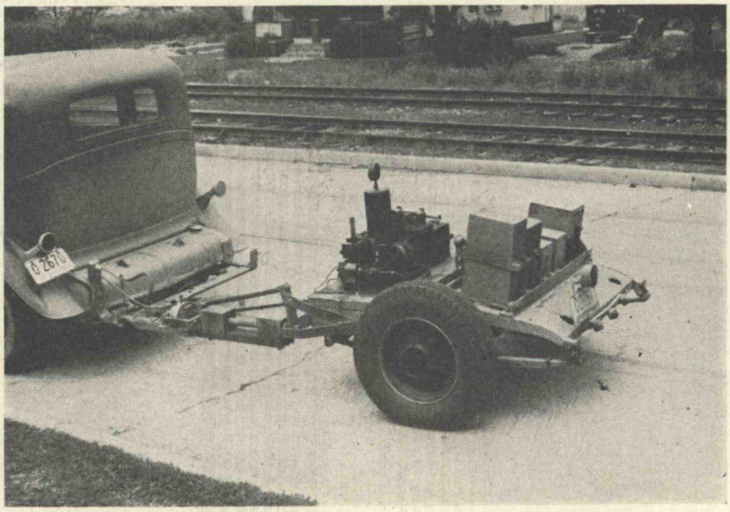

Figure 5. Test Car and Trailer

the left wheel so as to obtain a direct pull on the test wheel. When the trailer is coupled to the towing car, the trailer wheels are offset about eight inches from those of the towing car, so that the test wheel does not follow the track of the towing car. A hydraulic brake was used on the wheel and was operated by a constant oil pressure maintained in a tank on the trailer. The brake was applied and released by electrical control from the towing car.

"The test wheel operates a centrifugal slide indicator which gives a record, through a solenoid, of the instant the wheel is locked.

"The hydraulic dynamometer is mounted on the trailer draw bar and connects with the rear of the towing car. The dynamometer consists of two units—the dynamometer diaphragm and plunger, and the roller-bearing support. The rubber diaphragm has an active area of approximately ten square inches. The dynamometer liquid is a 50 per cent mixture of glycerine and alcohol. The roller bearing support for the dynamometer cylinder fits around the trailer draw bar shaft.

"The diaphragm type dynamometer has been installed recently in place of a two-inch cylinder and lapped piston. This has eliminated the possibility of any sticking or leakage, and the performance is much more consistent.

"The recording and controlling mechanism is mounted in the rear of the towing car. The brake is controlled through electric switches and a solenoid which operates the adjustable brake valve. The recording mechanism consists of an electrically operated recording drum over which sensitized paper is passed at approximately two inches per second and is rerolled after leaving the drum. Two records are made on this paper, the unit pressure in the hydraulic dynamometer, which is recorded by a conventional engine indicator unit, and the instant of sliding, which is recorded by a separate stylus operated by the slide indicator on the test wheel shaft. A base line is drawn by a third stylus for use in analyzing the record."

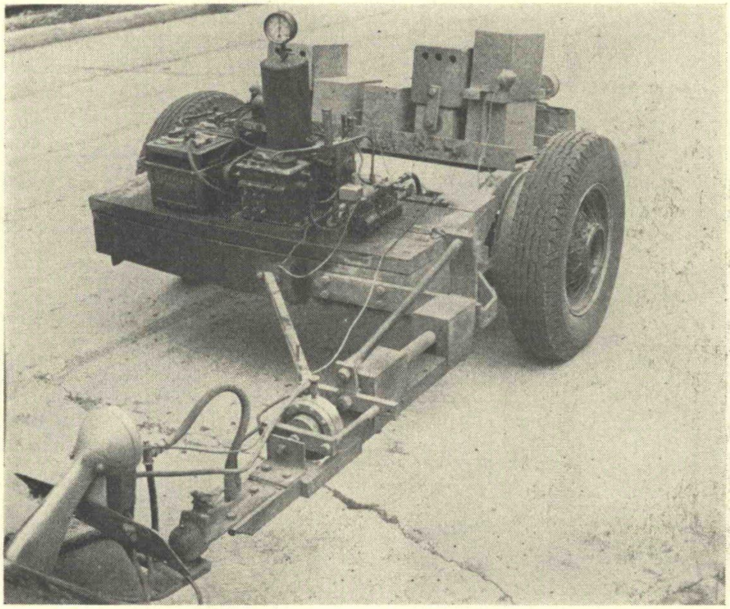

Figure 6. Trailer

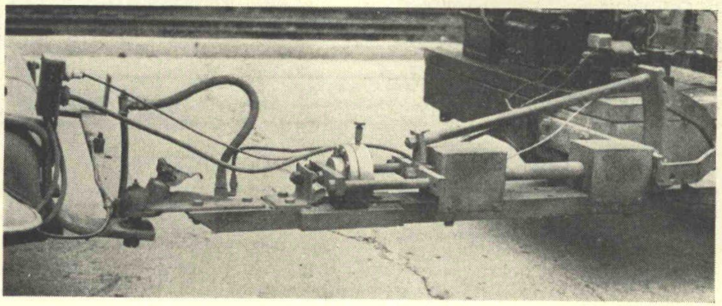

Figure 7. Hydraulic Dynamometer

Side Skid Apparatus: A description of apparatus for measuring side skid friction is given by Professor Stinson as follows:

"During the past winter, we have constructed a new trailer with which we are able to measure the side skid coefficient of friction. I am enclosing some photographs (Figs. 9, 10, and 11)

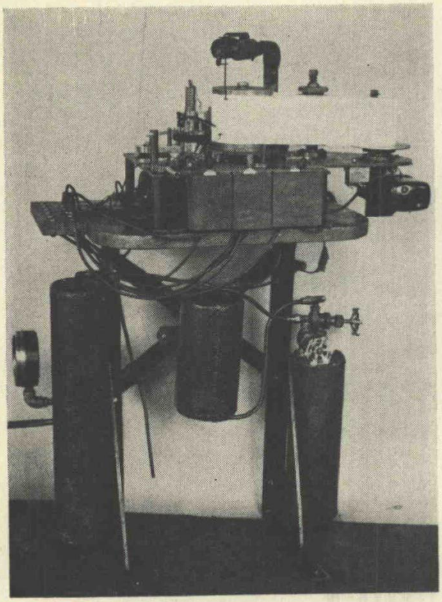

Figure 8. Recording Mechanism

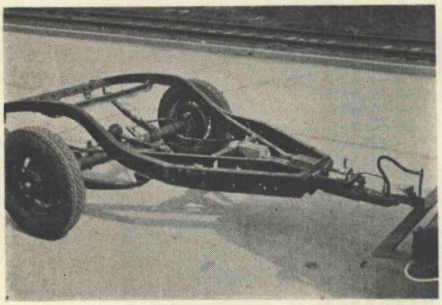

Figure 9. Trailer

of the equipment which were taken before the load was mounted. A normal tire load is placed on the frame while testing. The trailer is connected to the towing car through the hydraulic dynamometer used in our previous testing work. Both wheels are swivelled to the axle and can be turned to point toward each other in order to create a side skid action. These wheels are turned while in motion by a conventional starting motor to the desired angle. With the same recording mechanism that we have used in our other work, we can first record the tare force of drawing the trailer with the wheels parallel, and then can turn the wheels to the desired angle and take a record of the force required to overcome the skidding of the wheels.

"This apparatus has been used for comparative tests of several tires and road surfaces using normal tire loads and inflation pressures, although none of the results have as yet been published. The results are much more consis-

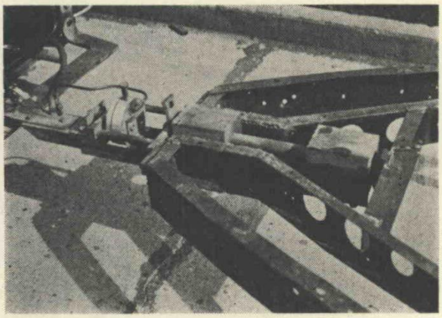

Figure 10. Hydraulic Dynamometer

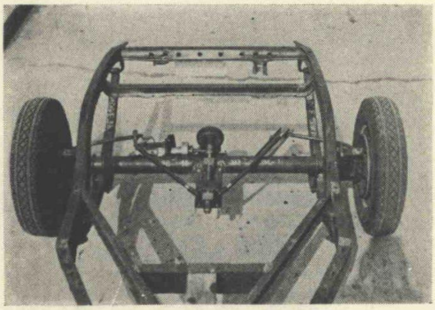

Figure 11. Device for Turning Wheels

tent than those obtained with the straight skid apparatus and can be made much quicker. Tests can be made with any angle of tow-in up to 15 degrees. The critical angle of modern tires carrying normal load is between 7 and 8 degrees. Any angle greater than this is satisfactory for test, and angles from 9 to 12 degrees have been used. Angles of 12 degrees or more have been found unsatisfactory due to (a) the abnormal scuffing of the tires even under short tests on wet surfaces, and (b) the great amount of power required to maintain uniform speed, particularly at high speed.

"All testing, either straight or side skid, is done with practically new stock tires. Consistent results can be obtained for 1,000 tests in straight skid with most tires, while in side skid they are used until the edges are well rounded over "The rolling or impending coefficient of a smooth tire is from 10 to 40 per cent less than a corresponding stock tire, and the variation on two roads will be much less, making comparative tests more difficult Smooth tires, unless free of all non-skid on the edges, will give variable results depending upon the amount of non skid present, and are going to be critical to small variations of tire pressure We feel that it is inadvisable to attempt to make comparisons of road surfaces with anything other than a comparatively new standard tire

"The side skid coefficients have been found to be about 30 per cent higher than the straight rolling coefficients on the same surface at speeds of 20 m p h and over This is thought to be due to the slow rate of relative sliding, and makes the values approach those of slow speed sliding"

APPARATUS USED BY THE MINISTRY OF TRANSPORT IN ENGLAND

This apparatus is fully described in Road Research Bulletin No 1 of the Department of Scientific and Industrial Research and Ministry of Transport, from which the following is abstracted

The apparatus at present in use consists of a motorcycle and sidecar, of which the sidecar wheel is mounted in a separate frame pivoted from the main chassis about a vertical axis, thus forming a castor arrangement as shown diagrammatically in Figure 12

"When testing a road surface, the motorcycle is driven along the road with the wheel fixed at an angle to the direction of motion, and both the normal force tending to restore it, and the load on the wheel, are measured The more slippery the road surface, the less is the force tending to push the wheel back into line The method of comparing the frictional properties of road surfaces therefore resolves itself into a comparison of the sideway restoring forces It is not this force alone, however, but the ratio of sideway force to the load on the wheel which has been selected as being the most suitable quantity for the comparison The resistance to slipping in a direction at right angles to the plane of the wheel is measured by this ratio, which is analogous to that generally known as the 'coefficient of friction,' but is called the 'sideway force coefficient' to denote its special nature

"Although the present design of machine is not perfect in all details, and while perhaps a motorcycle and sidecar will not necessarily remain the best form of instrument, the description of the machine now in use provides information regarding a satisfactory testing machine, with five years of practical experience behind it

Choice of Sidecar Wheel Angle "The sideway force coefficient is independent of the angle at which the sidecar wheel is set, provided the latter exceeds a certain critical value which depends upon tire characteristics

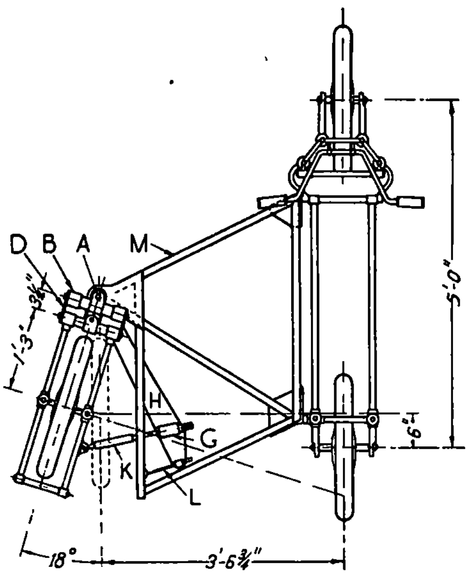

Figure 12 Diagrammatic Plan of Motor-Cycle and Sidecar Used by Ministry of Transport, England

"It was found from preliminary experiments that for comparative measurements on road surfaces, this angle should be 20° or more Actually an angle of 18° has been adopted throughout, as it was found that any increase in this value made the machine somewhat difficult to handle, and it was not considered expedient on the grounds of general safety to exceed it This compromise, however, only affects the highest values of the coefficient, and as these are the least interesting from the point of view of skidding, the small reduction from their true values is of no significance

Tyres "Since the machine is used to compare road surfaces, the tyre tread on the test wheel has to be of standard form It has been found that the usual tread patterns affect the results according to the amount of wear, and tyres with smooth treads are therefore used. These could not be obtained commercially, and a compromise was eventually found possible in the use of tyres of standard manufacture with the tread ground smooth. The form of tread is best shown on the left-hand wheel seen in Figure 14. Such tyres have been used for all routine skidding tests with satisfactory results; their only disadvantage being their short life due to the small amount of rubber on the tread. A sidecar tyre lasts, on the average, for about 30 miles of testing on wet surfaces, or

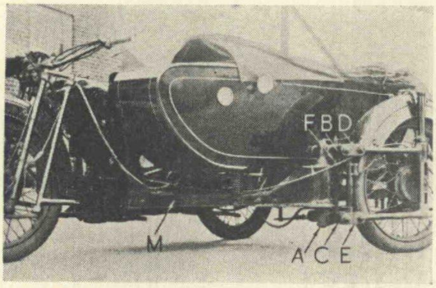

Figure 13. Motor-Cycle and Sidecar, Ministry of Transport, England

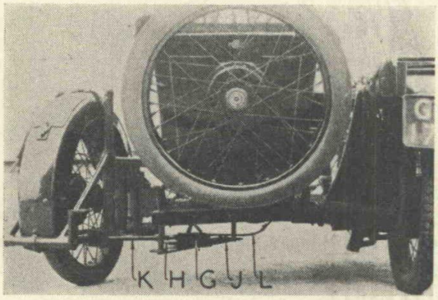

Figure 14. Motor-Cycle and Sidecar, Ministry of Transport, England

12 to 15 miles on dry surfaces. Wear when running normally as an ordinary motorcycle and sidecar is negligible by comparison. The inflation pressure adopted as standard was 50 pounds per square inch. The tyres used have so far been of the beaded-edge type, size 26 inches by 3 inches, fitted with security bolts. Arrangements have now been made with a tyre manufacturer for the production and supply of special wired tyres of the above size with a completely smooth tread of normal thickness to give increased life. A tyre with a smooth tread gives coefficients which are lower than would be obtained with tyres having treads in good condition. Since, however, tyres are often used until they are smooth, the results indicate the coefficients which are effective under the most disadvantageous conditions as regards tyre equipment in use on roads.

Disposition of Wheels: "The relation between wheel track, wheel base, and forward position of the sidecar wheel should be substantially as shown in Fig. 12.

Wheel Load: "When testing, the wheel loads of the machine now in use are approximately as follows: Front, 440 pounds; rear, 700 pounds; sidecar, 360 pounds. (These loads are for sidecar wheel in its testing position and with the driver in the saddle and the observer on the pillion.)

"It is apparent that the method of use described imposes a severe and complex loading upon the chassis of the machine. A chassis of usual design is insufficiently rigid to withstand this loading, and unless attention is paid to this point, severe oscillation of the machine is apt to arise, and chassis members are liable to break.

Sidecar Wheel Mounting: "The construction of the wheel mounting is best shown in Figures 13 and 14. It is free to pivot about the vertical spindle A, carried on the main chassis; at the top and bottom of the vertical spindle are horizontal lugs B and C, to which are pivoted links D and E supporting the sidecar framework. The load on the wheel is supported by the dynamometer F, consisting of a plunger $\frac{7}{8}$ inches in diameter working in a cylinder and transmitting the load by oil pressure to the recording apparatus. Similarly, the sideway force acting on the wheel when the machine is travelling forward with the wheel swung out, as shown in Figure 12, is measured by a second dynamometer of the same size, G (Fig. 14). The sideway force dynamometer G is carried on a plate H attached to the sidecar wheel pivot, the plate being supported by a second plate J bracketed to the main chassis. The sideway thrust on the sidecar wheel is transmitted to the dynamometer by a strut bar K, and the reaction on the dynamometer is taken by a Bowden cable L attached at its other end to a lever mounted between the motorcycle and sidecar which allows the angle of inclination of the wheel to be adjusted. The lever is provided with a trip which can be released from the handlebars of the machine in case of emergency. It has never been found necessary to use this safety device, but it is thought advisable to have it available.

Dynamometers and Recording Apparatus: "The variations in oil pressure, due to variations in load and sideway force while the machine is in operation, are transmitted through flexible metallic pressure tubing from the dynamometers to recording plungers, $\frac{1}{4}$ inches in diameter, controlled by springs which have a rate of about 45 pounds per inch. They extend as the oil pressure is increased, and this extension is recorded by mechanism described later. The complete apparatus is shown at N, Figure 15. The sideway force coefficient obtained from these spring extensions is continuously recorded on a paper roll, $8\frac{3}{4}$ inches wide, driven by clockwork. Synchronously, (a) time in $\frac{1}{2}$ seconds, (b) revolutions of the front wheel, and (c) signals as required by the observer, such as when passing from one type of surface to another, are recorded on the same roll by electromagnets supplied with current from the 6-volt lighting battery.

"Since, in order to obtain the least possible frictional lag, no packing of any sort is permissible between the dynamometer and recording plungers and their cylinders, great care is required in their manufacture in order to ensure free movement without loss of oil. The plungers are of mild steel, case-hardened, carefully annealed to remove internal strains before final hardening, grinding, and polishing to make their bearing surfaces parallel, and truly circular to give a maximum clearance of 0.001 inches and a minimum clearance of 0.0005 inches. Three shallow oil grooves are provided on the plungers. The cylinders are of phosphor bronze machined to the above limits.

"Since a small amount of oil leakage is necessary, the losses are made good by means of an oil gun having a screwed connection to lubricating nipples; these are isolated from the flexible pipe by means of a cock, which is opened only when charging the system with oil. A small, screw-down needle-valve is connected in each pipe to the recording plungers for the purpose of adjusting the damping according to the viscosity of the oil. By this means, large and sudden variations in the recorded pressure are smoothed out, while the comparatively slow changes due to variations in the surface are recorded.

"As already stated, it is the ratio of sideway force/load, called the sideway force coefficient, which is taken as the quantity by means of which the frictional properties of the road surface are compared. For convenience in evaluating records, the recording mechanism is designed to combine the two measurements of sideway force and load, so as to record the sideway force coefficient direct on the chart.

"The construction of this mechanism is shown in Figure 16. The oil pipe from the load dynamometer is connected at A, and the load measured by the extension of spring C; similarly the sideway force dynamometer is connected at B, and the force measured by spring D. The end of the load-measuring spring is connected through a link to E, which slides in guides F: similarly the end of the spring for measuring the sideway force is connected to part G. Part E is forked and G slides within it, being guided by slots in the same guides F. Parts E and G are arranged to slide in directions at right-angles to each other, and each is provided with a slot cut at right-angles to its direction of travel. Pin H passes through both slots where

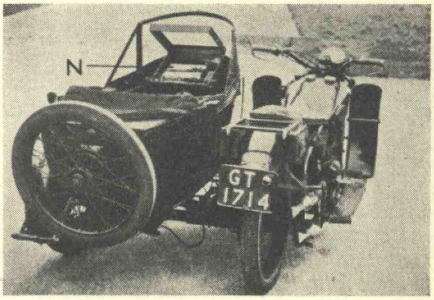

Figure 15. Motor-Cycle and Sidecar, Ministry of Transport, England

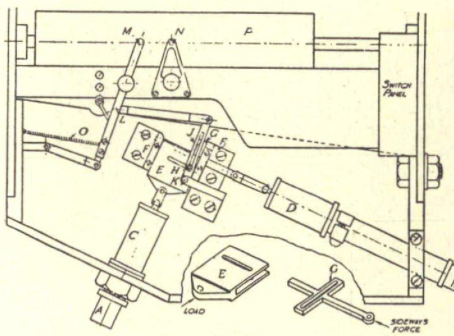

Figure 16. Coefficient Recorder, Ministry of Transport, England

their centre-lines intersect, and through a third slot in forked link J, pivoted at K.

"Link J is attached to the pen arm L carrying the pen M, by means of a straight-line linkage. A light spring O serves to take up the slack, while N is the fixed datum pen, and P is the drum over which the recording paper passes.

"It will therefore be seen that the displacement of M follows the angular movement of link J, which is in turn determined by the displacements of H in two directions at right angles, one proportional to the load, and the other to the sideway force Provided K coincides with the position of H for zero load and zero sideway force, a simple geometrical analysis shows that the displacement of the end of link J (and therefore also of the pen M) de-

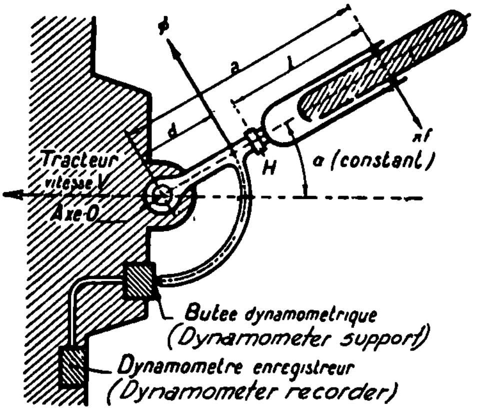

Figure 17 Diagram of the Boutteville Apparatus

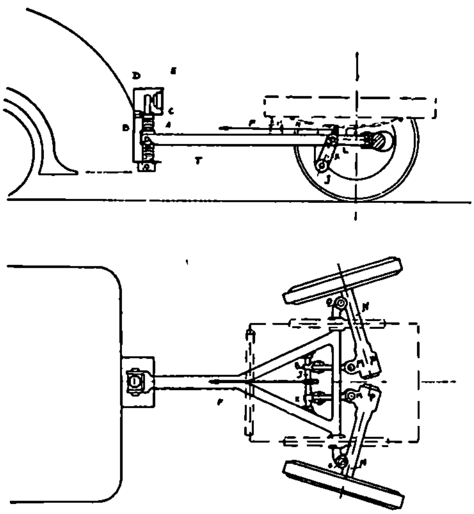

Figure 18 Diagram of the Bouly Apparatus

pends upon the ratio sideway force load or sideway force coefficient

"For a value of coefficient of 1, pen M moves to the extreme left, and, provided the loads per unit extension of the springs C and D are approximately the same, as they should be, link J takes up a position at 45° to the axes through K parallel to those of E and G"

APPARATUS USED BY BOUTTEVILLE IN FRANCE

This and the succeeding two types of apparatus are fully described in La Glissance Des Routes Et Sa Mesure published by the Syndicat Des Fabricants D'Emulsions Routieres De Bitume in Paris, France, from which the brief descriptions have been adapted

This apparatus is of the single wheel trailer type set at an angle to the line of travel The illustration, Figure 17, shows its construction

APPARATUS USED BY BOULY IN FRANCE

This apparatus is of the trailer type with two wheels set at an angle to the direction of travel Figure 18 shows an outline of the arrangement of the apparatus

STRADOGRAPHE DEVELOPED BY THE SYNDICAT S F E R B IN FRANCE

This apparatus is of the trailer type with two wheels set at an angle to the line of travel Figure 19 shows the arrangement of the apparatus

OPERATION OF SKID TESTING APPARATUS

Tires Tread design of tires has a definite effect on the test results Tires with non-skid treads wear down during the testing period and give varying results Some investigators correct this difficulty by using new tires and discarding them as soon as they become worn, while others use a special type of smooth tread tire

Condition of Surface Tests of dry surfaces are of little value Almost any pavement which has no loose material on its surface is skidsafe when dry

During a rain is the best time for testing

English results as reported in Road Research Technical Paper No 1 for tests on a concrete surface at 20 miles per hour are as follows:

	Coefficient
Road dry	0.85
Rain just starting	0.71
Surface first wet	0.49
Raining heavily	0.55–0.61
Road drying	0.51–0.61–0.76–0.85

The road may be sprinkled at the time of testing and a wet surface artificially produced. The coefficient obtained on a surface of this sort will be somewhat less than that for the same surface during a hard rain. This is not objectionable, since the worst conditions are being looked for.

English investigations indicate that about two and one-half Imperial gallons of water per square yard is needed properly to wet the road surface. This is three U. S. gallons. Professor Stinson comments as follows:

"When tests are made on wet surfaces with either trailer, it is always during a hard rain or with the test road continuously flushed and sprinkled by a street flusher. A strip of road surface approximately ¼ mile in length, where possible, is selected for test. It has been found that the condition of the surface is variable during the initial wetting. As a result, it has been found necessary to wash the surface several times in the spring and sometimes in the autumn, even half a day in order to get uniform conditions. This requires from 10,000 to 20,000 gallons of water per day. Our experience has led us to believe that it is practically impossible to obtain results of value by simply wetting the road ahead of the test tire."

Seasonal Effect: Road Research Technical Paper No. 1 states that there is a distinct drop in the value of the coefficients recorded during the summer months over those obtained during the winter months in England. Professor Stinson reports:

"For four years we have tested one section of a portland cement concrete road each April and September. Two conclusions have been that the coefficient of friction (a) decreases from year to year, and (b) is less in the autumn than in the spring. The later agrees with tests made in England, but we find that if the road is flushed for several hours, a large part of this decrease is eliminated."

Speed: Results obtained at low speeds are of little value. All investigators agree that the coefficients decrease as the speed increases. It would seem, therefore, that the testing apparatus should be operated

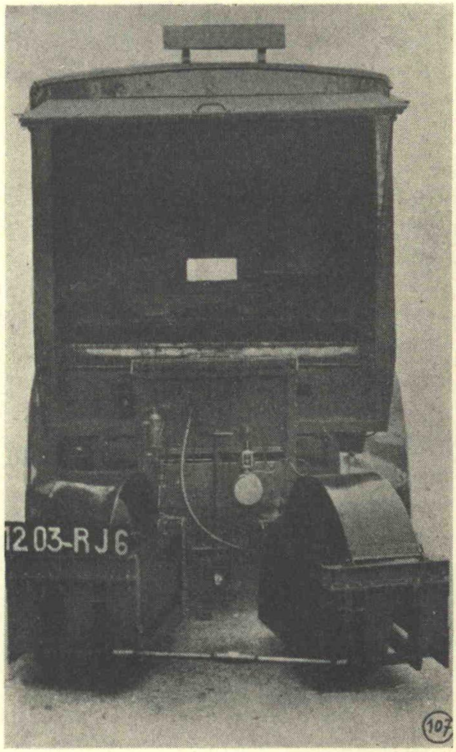

Figure 19. The Stradographe of the S.F.R.B. Syndicate Ready for Use (the Transverse Shock Absorber Has Been Placed in the Rear).

at as high a rate of speed as possible. This will probably be from 30 to 40 miles per hour.

CONCLUSIONS

Danger spots in a road system can be readily detected by the routine testing of highways for the determination of the coefficient of friction between the tires and the road.

The trailer type of testing apparatus possesses several advantages for this work.

Several satisfactory pieces of apparatus have been designed and tested sufficiently to insure satisfactory results

Either the longitudinal coefficient or the sideways force coefficient is satisfactory for the determination of the relative slipperiness of road surfaces

No check tests have ever been run over the same piece of road by the various kinds of testing apparatus However, a comparison of published results seems to show a fairly close check

Tests of types are not sufficient Each individual section of road must be tested

Tires used should be of uniform tread design and composition, and should be kept inflated to a uniform pressure

Tests should be run at the highest practicable speed, certainly not less than thirty miles per hour

Pavements should be tested when wet

Bird and Scott, in Road Research Bulletin No 1, speaking of conditions in England, say the following concerning the sideways force coefficient

"Values of 0 5 or more at 30 m p.h may be regarded as safe, whilst a value of 0.2 or less indicates a surface needing alteration According to conditions of traffic and location, values between these limits may be safe or dangerous "

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DISCUSSION ON MEASUREMENT OF SKID RESISTANCE

Mr J H Swanberg, *Minnesota Highway Department* It is apparent that skid resistance cannot be definitely correlated with type of surface The tests seem to confirm the conclusion that the surface types can be grouped only in a general way with respect to skidding coefficients Apparently any given section of road is dependent more upon the job control of the proportions and the ingredients than on the general type

Assuming the validity of the above conclusion, one can appreciate the need for established methods and apparatus for the making of friction tests on road surfaces If sideway skidding can be used for the basis of comparison, the problem of apparatus will be simplified, since it requires lighter and less complicated equipment and therefore greater mileage can be covered at less cost

Some argument has been presented that a driven wheel would give a different brake action than a non-powered wheel Would it be possible to have a driven wheel arrangement in addition to sideway adjustment, and would such an arrangement be practicable?

From the statements made concerning the results of wetting the road surface in the English investigation and in the comments of Professor Stinson, there would appear to be some question concerning the effect of sprinkling the surface immediately ahead of the test trailer as was done in the Iowa experiments We all know that a road surface at the beginning of a light rain is usually more slippery than after it has been rained upon for some time This is probably due to the presence of dust, which upon being wetted becomes slippery but is subsequently washed off

Has any work been done to show the probable errors of test results in order to arrive at the number of tests which should be made to yield a reliable average? Because of the small range of variation in skid coefficients, it would appear that a large number of test determinations should be made

It was shown by Moyer in his report "Skidding Characteristics of Road Surfaces" at the 13th Annual Meeting of the Highway Research Board that the majority of cars are not able to take advantage of available braking resistance when the coefficient is above a certain value, therefore it would appear that there should be an optimum skidding coefficient beyond which it would not be desirable to go because of the greatly increased tractive resistance and high tire wear

If a simple apparatus can be designed so that a large number of Highway Departments could avail themselves of the equipment, a comprehensive cooperative research project could be undertaken Such a research project would appear to be very desirable Any skid resistance policy will certainly have an effect on many industries in the highway field and highway engineers should have comprehensive data to justify any changes in highway surfacing types from the standpoint of skid resistance

Mr George E Martin, *The Barrett Company* The Committee did not feel that it was in position to design a specific piece of apparatus as suggested by Mr Swanberg but did feel that several types of apparatus were available and that there was no reason why individual organizations should not go ahead with the routine testing of existing pavements

It would, of course, be desirable from a research standpoint to have all of the pieces of apparatus the same, but it is not necessary to wait for that before starting to collect test data From our experience with engineering organizations in the past, we doubt that it would be possible to get any great number to adopt one specific type or piece of apparatus

I agree with Mr Swanberg that a skidding coefficient which is too high may increase tire wear and be objectionable from that standpoint, but I do not believe that it would increase tractive resistance

LABORATORY APPARATUS FOR FRICTION TESTS BETWEEN TIRES AND PAVEMENTS

BY ELMO E. HANSON

Technical Physicist, Mines Experiment Station, University of Minnesota

SYNOPSIS

A machine has been built to measure in the laboratory the following characteristics of iron paving blocks: (1) the coefficient of friction between the paving blocks and rubber tires for straight skidding and for sideways skidding, (2) the stopping distance of a car on such a pavement, and (3) the tire noise. The machine consists of two six-foot flywheels, on the periphery of which are bolted the paving plates to be tested. An automobile is mounted such that its front wheels rest on these flywheels. The flywheels are rotated by a 15 horsepower motor to top peripheral speeds of 60 miles per hour. Hydraulic traction dynamometers are connected to the center of gravity of the car and serve to measure the braking force and the sideways force when the brakes are applied or when the front car wheels are turned through an angle. A high-speed motion picture camera is used to record the dynamometer readings, the time, and the motion of the flywheels and of the front car wheels. Examples of the curves so obtained are given. It has been found that by changing the design of the surface pavement corrugations, the coefficient of friction may be changed by more than 100 per cent.

If cast iron is to be used as a surfacing material for pavements, it is imperative that it have a corrugated or studded surface. While rubber tires on wet, polished, smooth iron have a coefficient of friction of only about 0.2, it has been found possible to increase this value greatly by using a roughened surface. Since iron can be cast into a large variety of designs, there is the possibility of developing a surface having very large coefficients of friction. In attempting to devise such a surface it was necessary to have a rapid and cheap method of testing pavement surfaces.

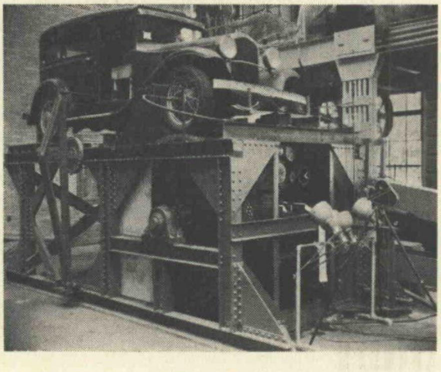

Figure 1

A laboratory testing machine was built to measure the following functions of the relative speeds of the pavement surface and automobile: (1) the coefficient of friction both for skidding and skidding impending; (2) the stopping distance of a car; (3) the coefficient of side friction (steering response); and (4) the noise generated by the interaction of the pavement surface with the automobile tires. While the absolute magnitudes of these quantities as measured in the laboratory may differ from those which would have been obtained if the tests had been made on a highway of the same material, the measurements serve as a basis for comparison of the experimental surfaces. It is planned to correlate our laboratory measurements with highway measurements later.

APPARATUS

Figure 1 is a photograph of the testing machine. It is essentially a device for moving the pavement surface while the test car is at rest. The cast steel paving surface is bolted to the periphery of two 6-ft. flywheels which are keyed to a 5-in. axle mounted in roller bearings supported by a heavy steel frame. The method of mounting the pavement plates is shown in Figure 2. By means of a 15 hp. slipring motor and a belt drive the flywheels can be given peripheral speeds up to 60 m.p.h. A 1931 Plymouth car is mounted so that its front wheels rest on the paving surface, while its rear wheels are supported by a platform. The car is held in place by two cables at right angles fastened to its center of gravity. The cables are about 3 ft. long and, therefore, permit the car to bounce up and down as it would under actual driving conditions. The master cylinder of the hydraulic brake system of the car is mounted near the bottom of the machine frame with a lever to operate it. Copper tubing and a short section of reinforced rubber tubing connect the master cylinder to the front brakes. The car wheels are made to track straight on the pavement by means of a lever attached to the tie-rod, pivoted at the axle, and held between two clip angles.

An attempt was made to simulate as nearly as possible the actual driving conditions on a highway. When an automobile is driven around a curve or when it is decelerated by the application of its brakes, the inertial force acts as though it were all concentrated at the center of gravity. For this reason, the restraining forces in the laboratory set-up are applied and measured at the center of gravity of the car.

The second condition of similarity to driving conditions to be fulfilled is that the kinetic energy of the flywheels at a peripheral speed v shall be equal to the energy normally dissipated at the front wheels of a car when it is stopped on the highway from a speed v. If we assume a car (with all four wheels locked) skidding to a stop from a speed v, the energy dissipated at each wheel is proportional to the weight on that wheel provided that the coefficient of friction is the same at all four tires. Kinetic energy is also proportional to weight. Therefore, it can be shown that in such a case the energy dissipated at each car wheel is equal to the initial kinetic energy of the weight supported by that wheel. Therefore, the second condition to be fulfilled by the test machine can be expressed by the equation:

$$\frac{1}{2} I \frac{v^2}{R^2} = \frac{1}{2} m_f v^2,$$

Wherein I is the moment of inertia of the flywheels, v is the peripheral speed of flywheels, R is the flywheel radius, and m_f

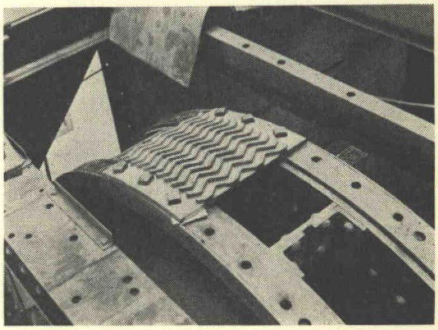

Figure 2

is the total mass supported by the front car wheels. Solving the equation for I, we have:

$$I = m_f R^2$$

However, in order to make the flywheels strong enough to stand the top operating speeds safely, it was found necessary to build the flywheels a little heavier than the weight obtained from the above equation. The only calculation in which I enters is that of stopping distance, and here the proper corrections for the discrepancy in I are made.

The iron pavement which has been in use on the University of Minnesota campus for the past four years has acquired a high degree of polish, and the edges of its surface lugs have been only slightly rounded. To make the test surface in the laboratory set-up correspond to a pavement which has been in use for several years, the following "aging" process is used:

After the pavement plates have been bolted to the flywheels, a light cut is taken across their surface with a lathe tool which is carried in a portable lathe carriage bolted to the testing machine (the flywheels being rotated by a motor through a speed reducer) (see Fig. 3). Next a No. 80 grit emery wheel is mounted in the lathe carriage, and the flywheel surface is ground to eliminate the scratches made by the cutting tool.

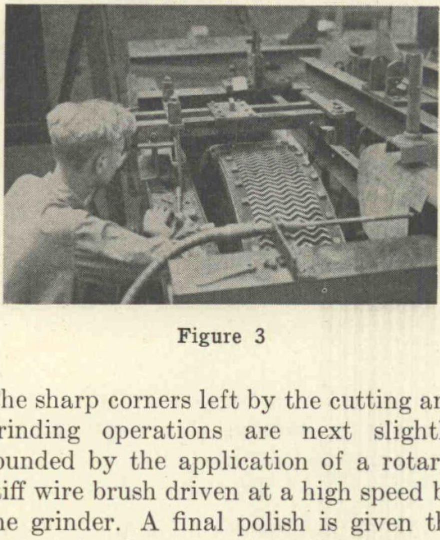

Figure 3

The sharp corners left by the cutting and grinding operations are next slightly rounded by the application of a rotary, stiff wire brush driven at a high speed by the grinder. A final polish is given the surface by the manual application of a fine emery cloth while the flywheels are rotated at a high speed.

All of the tests are run on a wet surface. Other investigators[1] have found that if a surface has satisfactorily high coefficients of friction when it is wet, it also will be satisfactory when it is dry. The water is sprayed on the surface at the rate of about $1\frac{1}{2}$ gal. per minute.

[1] Moyer, *Proceedings,* Highway Research Board, Vol. 13, p. 123 (1933) and Stinson & Roberts, p. 169.

Both new and old tires are used for all of the tests. The procedure is generally to use a new nonskid tread tire until it is slightly worn, and then to cut off its tread to make it correspond to an old tire. The tire tread is cut off on a lathe by the use of a sharp, flat knife which is constrained to move in a special templet to give the correct curvature to the cut. This curvature was determined from measurements on several old tires. The tire size is 4.75 by 19 in., and the tire pressure used is 35 lb. per sq. in.

MEASURING INSTRUMENTS

For the computation of the coefficients of friction, the braking force and the side force acting on the car are measured through the anchoring cables by hydraulic dynamometers, which were designed and constructed in this laboratory. The side cable-lever-dynamometer set-up can be seen in Figure 1. Each dynamometer consists of a bourdon tube pressure gauge connected through a $\frac{3}{4}$ in. copper tube to a water-tight steel drum, one side of which is thin, flexible steel sheet. When the dynamometer is filled with water, it will accurately measure forces applied to the flexible face of the drum. The effective area of the dynamometer drum is about 100 sq. in., and since the displacement of the bourdon tube gage is less than $\frac{1}{2}$ cu. in. for the maximum force recorded, the motion of the drum face is less than 0.005 in. Therefore, the measurement of the brake force and of the side force requires only a negligible motion of the center of gravity of the car. This is, of course, necessary in order to have as small a time lag as possible in the force measurements.

The rotation of the flywheels and of the front automobile wheels is indicated on special "clocks" made in this laboratory. These revolution indicators consist of an 8-in. dial with pointers driven through a gear box The flywheel indicator is coupled to the flywheel axle through a chain drive and can be read directly to 0 025 revolution Each of the automobile wheel indicators is coupled to the automobile wheel through a flexible shaft, and can be read to 0 1 revolution

Time is measured by a revolution indicator of the above type, driven by a 60 cycle synchronous motor Time can be read directly to 0 025 sec and estimated to 0 010 sec

A standard automobile speedometer is coupled through a belt drive to the flywheel revolution indicator It gives the operator the approximate speed of the flywheels when tests are being made More exact determinations of the speed are made from the data on flywheel motion and time

A pressure gauge indicates the oil pressure in the hydraulic brake line

Two recording vibrographs were built to show the vertical and horizontal motion of the automobile axle The recording mechanism is driven by the same synchronous motor that marks the passage of time, so that the vibrograph readings can be correlated with the other data

All of the instruments are mounted on a 3 by 4-ft panel set in the front of the machine, as shown in Figure 1 This permits a motion picture record to be made of the instruments during a test

A 16 mm Eastman Ciné-Special motion picture camera is used to record the readings on the instrument board For most of the work the camera is operated at 16 frames per sec and ¼ shutter opening to give an exposure of 1/128 sec This "stops" the motion of the pointers quite well In some of the work, camera speeds as high as 64 frames per second are used The negative panchromatic film is viewed in an Eastman "Recordak," and the instrument readings are transcribed to data sheets

TEST PROCEDURE AND CALCULATION

Coefficients of Friction: The two brakes are first equalized so that when the hydraulic brake pressure is gradually increased the two wheels lock at as nearly the same instant as possible This adjustment is always made before the coefficient of friction tests are made

The test routine is as follows: The flywheels are brought up to the desired speed by the motor, the power is shut off, and one operator applies the brakes while the other operates the movie camera The brakes are left on until the wheels have been locked for about 1½ sec The tires are then washed with cold water (for cooling and for carrying off the small rubber particles that have been rubbed off), and the same procedure is repeated for another speed Usually tests are made at the lower speeds first, because the greatest tire wear occurs at the high speeds Tests are run from 5 to 60 m p h at 5 to 10 m p h intervals

Next the brake force dynamometer is calibrated under the same conditions as those under which the tests were made A rod is attached to each end of the front axle, and it is connected at the other end to the vertical arm of a right angle lever which is pivoted at its apex on the machine frame To the horizontal end of the lever are hung the calibrating weights The levers have a mechanical advantage of 5, so that the horizontal force applied to the car is equal to the total calibrating weight multiplied by five Turnbuckles in the connecting rods serve to take up the "give" in the front springs when the force is applied The readings of the dynamometer plotted against the corresponding known forces give the calibration curve This calibration curve has been found to be a straight line

After the film has been developed and the data transcribed, the dynamometer readings are plotted against the time. A typical curve is shown in Figure 4. The peak in the curve is the value of the frictional force for skidding impending, while the average force over a ½ sec. interval, beginning ¼ sec. after the peak position, is taken as the value of the frictional force for the wheels locked. From the calibration curve, the frictional force in pounds is calculated. The corresponding speed of the pavement surface with respect to the car is calculated from the flywheel-time data.

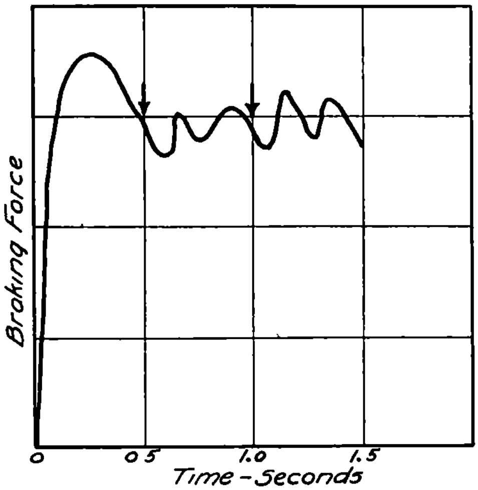

Figure 4

The coefficient of friction is calculated from the following formula:

$$u = \frac{F}{W + 0.198F}$$

Wherein F = frictional force
W = static load on the front wheels of the car.

The factor 0.198F in the denominator is the increase in the weight on the front wheels of the car when the force F is applied. It has been shown by Moyer [1] that due to the couple acting on a car when the brakes are applied, the effective weight on the front wheels is $W + \frac{HF}{L}$ where H is the height of the center of gravity, and L is the wheelbase.

It was found that when a braking force was applied, the car wheels would be forced back a little due to the elasticity of the springs. This displacement was never more than ¼ in. It can be shown that if the car wheels are displaced by this amount, the increase in the observed coefficient of friction is to the order of 0.005 or less than 1 per cent

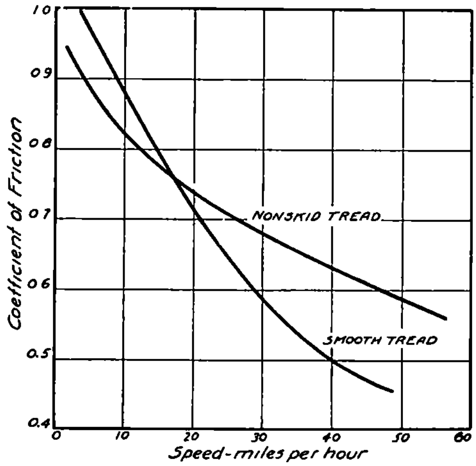

Figure 5

of most measurements. The effect was compensated for, however, by initially setting the car so that the front wheel centers were $\frac{3}{16}$ in. ahead of the vertical through the flywheel centers.

The resulting coefficients of friction were plotted as a function of the flywheel peripheral speed in the usual manner. Such curves are shown in Figure 5.

Stopping Distance. The test procedure for determining the stopping distance is as follows. The flywheels are brought up to the desired speed, the power is cut off, the brakes are applied to lock the car wheels, and the flywheels are thereby brought to a complete stop. A motion

[1] Moyer, *Bulletin* No. 120, page 76, Iowa Engineering Experiment Station.

picture record over the same range of speeds as before is made of the instrument board.

From the motion picture record, the initial speed v and the distance d travelled by the periphery of the flywheel are determined. The equivalent stopping distance of a car is calculated from the formula:

$$d_c = \frac{WR^2}{I} d + \frac{0.198v^2}{2g}$$

Where g is acceleration of gravity, v is the initial speed in feet per second, W is the static mass supported by the front car wheels, R is the radius of the flywheels, and I is the moment of inertia of the flywheels. This serves to reduce all stopping distances to the same moment of inertia, and also to approximate the stopping distance of a car on a highway of the same surface as that tested.

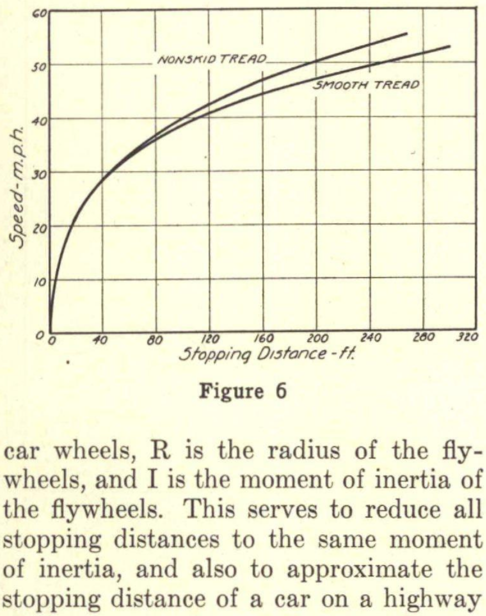

Figure 6

The stopping distance is plotted against the initial flywheel speed. This curve is, then, a characteristic of the pavement surface. Figure 6 is an example of such a curve.

Coefficient of Side Friction (*Steering Response*): For the tests on side friction, the clip angles are removed from the automatic steering device in order that the car wheels may be turned through some steering angle ϕ. The steering apparatus is equipped with a stop that can be clamped at any desired steering angle within the range of the steering gear of the car. A picture of this set-up is shown in Figure 7.

The following procedure is used for making the tests: The clamp on the steering mechanism is set at the desired steering angle ϕ, the flywheels are brought up to the desired speed v, and the power is cut off. One operator turns the steering wheel until the stop is hit, while the other operator gets the motion picture record of the instruments. After about a second, the wheels are straightened and the procedure is repeated for some other speed. Generally for each angle ϕ tests are made at 10 m.p.h. intervals up to 60 m.p.h. This is done for steering angles over the range available (about 22 deg.) at about 4 deg. intervals.

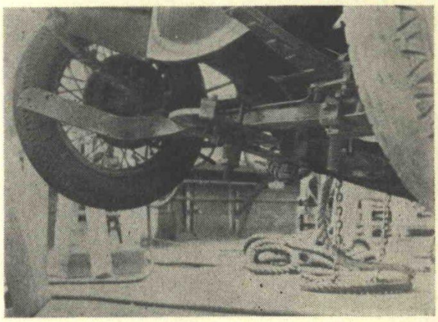

Figure 7

This side force dynamometer is calibrated for the conditions under which it is used. The known calibrating force is applied in line with the front axle by a lever-weight system, similar to that used for the brake-force calibration. In this case each front wheel is made to rest on two horizontal plates with rollers between in order that they may be free to move. The tires are deflated enough to bring the axle down to its normal level. The readings of the dynamometer plotted against the known force gives the calibration curve and serves as a basis for the calculations of the side force of friction It, too, is a straight line

From the motion picture record of the side force dynamometer, the average force acting is calculated for the angle ϕ, and the speed v which is determined as before The coefficient of side friction, or the steering response, is then

$$u = \frac{\text{force of side friction}}{W}$$

A small correction is applied to this value to compensate for the effect of the displacement of the car wheels when they are turned through an angle Figure 8 shows the type of curve obtained for side coefficient as a function of speed

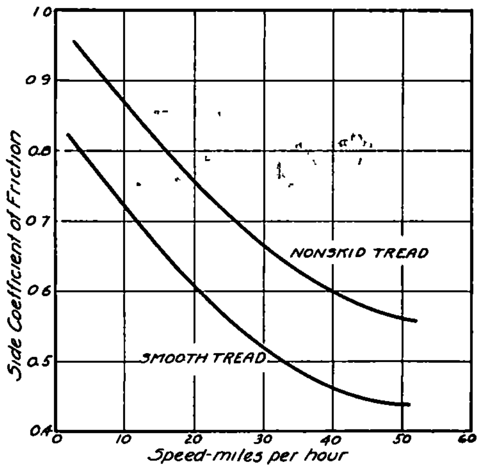

Figure 8

There were some experimental difficulties in making these measurements due to the flexibility of the car and its tendency to tilt under a side force The tilting was largely eliminated by introducing a torsional stabilizer at the rear springs The difficulties due to the flexibility of the car were reduced by tightening the side dynamometer cable to introduce an initial side force on the car somewhat less than that to be measured, the car being pulled up against a stop When the front wheels were turned through an angle, the car would pull away from this stop, so that the force registered was the one actually acting at the front wheels In this way it was possible to cause the car wheels to track near the center of the rotating pavement even under very high side forces The dynamometer was always calibrated under the same conditions as it was used

Noise Measurements A General Radio Type 559B noise meter was used to measure the noise generated by the tires on the pavement surfaces The noise meter is set about 20 in from the tire and pointing toward the point of contact between tire and pavement Readings are then taken as a function of the flywheel speed The background noise is next measured as a function of speed with the car jacked up so that the tires are not in contact with the pavement The noise intensity in decibels is plotted against velocity, both for the background and for the tire noise The difference between the two curves gives the noise due to the interaction of tires with the pavement Figure 9 is an example of such a noise

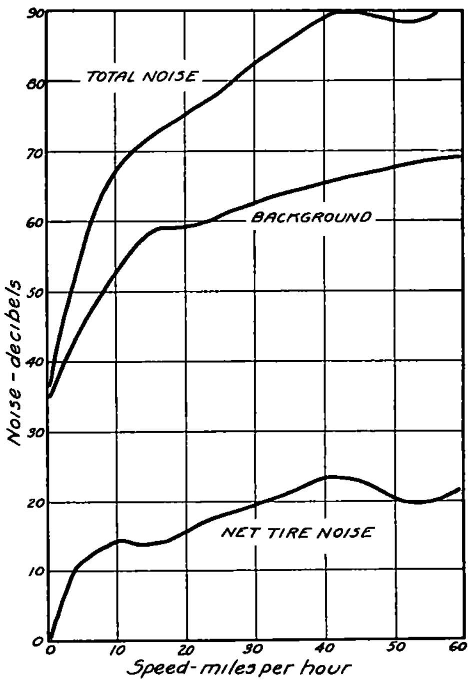

Figure 9

curve Similar procedure is used to obtain the noise inside of the car Since the background noise is substantially the same for all of the test surfaces, the method indicates the relative "noisiness" of the pavements tested

CONCLUSION

Very marked changes can be produced in the coefficients of friction of tires against a wet, cast steel pavement by simply altering the design of the pavement surface Seven designs of a roughened surface have so far been tested, and the range in value of the coefficients for these surfaces is given in Table I

It should be emphasized again that these values may not be the same when they are measured on a flat surface The correlation of the values as measured on a flat surface (a highway), and the values as measured on the curved surface in the experimental set-up, is to be carried out this summer (1938) An iron pavement one-half block long is at present being laid on a campus street, and the identical design of surface used on this street is also being tested in the laboratory It is hoped that by comparing the values of the coefficients of friction and the stopping distances as measured on the street

TABLE I

(All values given for 30 m p h peripheral speed of flywheels, and for a wet surface)

	Coefficients of friction		Coefficient of side friction	Stopping distance, feet
	Skidding impending	Wheels locked		
Nonskid tires				
High	83	70	91	90
Low	46	34	49	44
Smooth tires				
High	86	66	75	104
Low	32	24	48	40

to those measured on the laboratory test machine, some conclusion may be reached as to the relation existing between the two methods of measurement More can be said about the absolute magnitudes of these values after the correlation tests have been carried out

REPORT OF COMMITTEE ON FLEXIBLE TYPE SURFACES

WHAT WE KNOW ABOUT THE DESIGN OF PAVEMENTS OF THE FLEXIBLE TYPE

By A C Benkelman

Associate Research Specialist, U S Bureau of Public Roads

(In Abstract)[1]

The report presents a digest and discussion of published material pertinent to the problem of design of flexible pavements

It points out to begin with that while our knowledge has progressed to the point where subgrades can be prepared more or less according to scientific principles, adequate information is lacking concerning their quantitative bearing capacity, at least that type of information suitable for use in design formulas The reason for this is that most of the investigations of bearing power have dealt only with the reaction of soils to sustained pressures beneath bodies, essentially rigid in character, such as foundation structures

The forces to which subgrade soils are subjected are radically different in character First, the imposed pressures are seldom purely static and second, road surfaces through which the pressures are transmitted possess more or less flexibility resulting in nonuniform pressure distribution

Evidence is presented in the report to show that pressure distribution may not only depend upon the character of the subgrade soil but to a very marked extent upon the degree of flexibility of the road surface through which the pressure is transmitted, and in view of this evidence the statement is made that it does not appear that we should attempt to apply our knowledge or conceptions of rigid footing stress distribution in soils directly to the problem of flexible pavement design

It is generally recognized that most soils deform less under a dynamic than under a static load of the same intensity However, definite information is lacking as to the extent of the difference in soil supportability under these two types of load As far as flexible pavements are concerned this is an exceedingly important question because here the subgrade soil constitutes practically the direct medium of load support inasmuch as the areas of subgrade pressure are relatively small and the unit intensities of pressure high In other words the more the factor of subgrade support enters into the performance of a road surface, the more necessary it becomes to take into account the dynamic properties of the subgrade soil

This question is discussed in considerable detail in the report and some space is devoted to a description of the German oscillator method of test[2] that appears to have promising possibilities for studying the dynamic properties of soils and road surfaces This method of test involves the application of periodic forces, the intensity of which varies according to a sine law, to a material and the study of its physical reaction by measurements of displacement and speed of propagation A machine, consisting essentially of two parallel shafts that rotate in opposite directions and to each of which is attached an eccentric weight is used to

[1] The complete report has been published in Public Roads Vol 18, No 11, January 1938

[2] See R K Bernhard, Discussion on "Stresses in Concrete Pavement Slabs," page 230

apply the forces The displacement of the soil medium or road surface is measured at different points with electrically operated instruments

According to a number of German Engineers the method of test can be used for developing quantitative information regarding the ability of road surfaces laid upon different types of subgrade soil to support loads of a dynamic nature, as well as for studying their vibratory properties

The fact that it is possible to apply repeated dynamic forces of known magnitude and characteristics to a road structure with an oscillating machine, is cited as one of the advantages of the method of testing It should be possible with such a machine to apply forces that are similar in character to those which are imposed upon pavements by moving vehicles inasmuch as the revolving eccentric weights are analogous to the unsprung mass of a vehicle and the weight of the machine is analogous to that of the sprung load

Another section of the report describes and discusses the methods of design of flexible pavements that have been suggested to date It is pointed out that these methods are premised more upon theoretical conceptions of soil resistance and pressure distribution than upon pertinent or comprehensive test data

The fact is mentioned also that before the required thickness of flexible pavements can be determined for a given set of conditions by any formula, the question as to what is the critical wheel load must be answered In other words, if, at the present time, a formula is used to calculate the thickness of surface, one must make some sort of an arbitrary allowance to compensate for the possible dynamic or impact effects that moving vehicles may have upon the pavement This is evident from the fact that in some instances an allowance of 50 percent over and above the permissible static wheel load is advocated In others no allowance is specified because it is contended that the dynamic resistance of flexible pavements, i e , the resistance that materials composing such pavements offer to deformation under quickly applied loads, may entirely outweigh any actual impact forces

In the summary of the report the major parts of the general problem that need attention are itemized They include

1 A study of the load supporting and pressure-distributing ability of typical surfaces of the nonrigid type as influenced by—

(a) The magnitude of the load
(b) The area of load application and the distribution of pressure intensity over the area
(c) The time duration of the load
(d) The physical characteristics of the surface course and of the subgrade

2 The development of data that indicate more directly the safe load-supporting value of soils when subject to forces and displacements such as obtain under road surfaces of the nonrigid type Factors that probably exert an important influence are—

(a) The size of the area of applied pressure
(b) The rigidity of the surface through which the pressure is applied
(c) The effect of restraint to vertical movement around the area of applied pressure
(d) The physical characteristics of the subgrade material

3 The determination of the relative effects of slowly applied and suddenly applied forces in order that the critical load for design purposes may be known

DISCUSSION ON FLEXIBLE TYPE PAVEMENTS

PROF D P KRYNINE, *Yale University* Mr Benkelman rightly states that it is not known whether the action of a static wheel should be taken into consideration in designing a flexible pavement or whether a dynamic force prevails, and this question is to be studied and discussed At this time there are quite a few formulas which permit us to design a flexible pavement The speaker wishes to emphasize the fact that all these formulas are based on the static approach to the problem

The principal difference between a static and a dynamic stress distribution through a flexible pavement is that in the former, forces are immobile, and the whole pattern is stiff and dead, while in the latter the stress pattern is living and moving as it is in reality If we could apply elastic formulas to a flexible pavement and to the earth mass underneath, we could compute displacements under a static wheel This problem treated as a three dimensional one is unusually complicated, but may be simplified considering it in a plane If the car is moving, it is questionable whether elastic formulas may be applied unconditionally If, for instance, the speed is forty or more miles per hour, displacements probably will not be the same as under a static wheel, and it is not even known whether stresses have time enough to penetrate into the interior of the earth mass The speaker thinks that this limitation as to the depth of propagation of stresses under a dynamic load is an important one, not only for the problem of pavements, but for railroad engineering as well ' The problem in question has never been solved

Displacements under a car take the form of a hollow (so-called "crater") in the flexible pavement, and at every given moment the car is at the bottom of this hollow Hence the car may be imagined as moving along a curved elastic line, and perhaps in this case centrifugal force acting in a vertical plane should be taken into consideration It is true that in an ideally homogeneous horizontal flexible pavement the action of this vertical centrifugal force vanishes, since after all the car moves along a horizontal plane But if there are inequalities within the body of the pavement or if the car moves up hill, the vertical centrifugal force may produce a certain effect Admittedly, the existence of such a centrifugal force should be proved by adequate research But if it exists, it certainly produces a much larger effect in a flexible pavement than in a rigid concrete pavement because of the smaller radius of curvature of the elastic line in the former In an analogous way, the weaker the earth mass under a flexible pavement, the deeper is the hollow under the car Hence comes the idea of reinforcing and stabilizing earth under a flexible pavement

In a really flexible pavement the car should press very strongly against the bottom of the hollow already referred to Hence the speaker does not believe very much in the possibility of a considerable impact action on flexible pavements The case of a rigid concrete pavement is different, because the smallest inequality on the surface of such a pavement causes the car to drop from a certain height to the level of the pavement with consequent development of kinetic energy It is difficult to visualize a similar action in a really flexible pavement, although it may partially develop Again this is a problem to be solved by adequate research

The speaker believes that stresses in a flexible pavement may be determined from actual displacements under the moving car, and to determine the shape and the size of this elastic line or rather elastic surface observations must be made Before this difficult job is undertaken consideration should be given to the problem of whether stresses can be determined in some other way One thing is clear, however static formulas and the angle of pressure distribution of 45° can hardly be of great help in this case

Finally in studying the stress distribution in a flexible pavement the gradual increase in pressure should be taken into consideration The stress at a point develops gradually as the car approaches It starts at a zero value when the car is theoretically at infinity, and practically at a certain distance from the given point The maximum value is reached when the car is exactly over the given point, and as the car goes away, the stress decreases in the same manner In this stress increasing-decreasing process the effect of vibrations may be of importance, and the speaker is concluding his discussion with a wish that German methods of studying vibrations as described by Mr Benkelman might be introduced in this country as soon as possible

Mr J A Buchanan, *Bureau of Public Roads* If I understand Professor Krynine correctly, he said that impact is not developed on flexible type pavements If by flexible type pavement you mean a bituminous mat laid on the subgrade, such as that commonly known as black top pavement, then I would like to say that impact is developed on that type of pavement We have measured the forces developed and, the impact forces are in general of the same order as developed on the rigid pavements for a given surface roughness

Professor Krynine The static load may be increased by the centrifugal force

Mr Buchanan Definite impact reaction is developed because of the inequalities on the surface of the road

Mr Rockwell Smith, *Minnesota Highway Department* Mr Benkelman has done a great amount of work in reviewing and reporting tests on soil loading, but there is nothing in these tests which can be translated into a mathematical treatment of a design problem The tests as described, however, indicate that our present empirical method of design is not contrary to established facts As Mr Benkelman mentions, we have the qualitative means but no method of applying these means in a quantitative manner

There is one point not clarified in Mr Benkelman's report He speaks of flexible surfaces and foundation courses without any special distinction as to their flexibility We have found that a good stabilized gravel base may be regarded as semi-rigid to a certain extent As a real flexible course, I would mention most bituminous treatments and water bound macadam such as we have in Minnesota To cite an example of my point, there is a 200 or 300 ft section south of Spring Valley on which the subgrade has displaced both laterally and vertically the latter displacement to an extent of 3 in Over this displacement the flexible macadam and bituminous mat has remained without breaking

It has been our experience that with much less displacement a stabilized gravel mat would have broken and disintegrated Or if thick enough its beam action would have bridged the area and prevented the displacement In the laboratory we have developed stabilized sand beams with a modulus of rupture of 200 lb and compressive strength of 600 lb per sq in in a 2-in cube

Thus various so-called flexible surfaces will vary greatly in their load distributing action and in their performance under displacement

The use of the numerous formulas

reported is not practical in our design work With the best available information these formulas give values of base thickness much greater than those we are using in our design As Mr Benkelman points out there is a great lack of information concerning the actual loading a pavement receives under moving wheel loads Also there is no adequate knowledge of the bearing capacity of subgrade soils He sums this up very completely on page 50 of his report

Mr Benkelman also points out that the methods described have contributed to our understanding of the problem, but that they are premised on conceptions of resistance and distribution rather than on actual data This is in complete accord with our thought of the methods described

REPORT OF PROJECT COMMITTEE ON THE USE OF HIGH ELASTIC STEEL AS REINFORCEMENT FOR CONCRETE

By H J Gilkey, *Chairman*

FOREWORD

This report constitutes Chapter VIII 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

References to the chapters published in the Proceedings of the Highway Research Board to date 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 Economics 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)

The work reported as Chapter VIII was conducted as a cooperative project between the Highway Research Board, and the Iowa Engineering Experiment Station The results of the tests were reviewed by the project committee of the Highway Research Board which is now constituted as follows

G C Ernst, Assistant Professor of Civil Engineering, University of Maryland
A L Gemeny, Senior Engineer, U S Bureau of Public Roads
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
S B Slack, Consulting Engineer, Decatur, Georgia
H J Gilkey, Professor and Head of Theoretical and Applied Mechanics, Iowa State College, Chairman

CHAPTER VIII. BOND RESISTANCE OF HIGH ELASTIC LIMIT STEEL BARS, SERIES OF 1937

By Herbert J. Gilkey, *Professor and Head*

Stephen J. Chamberlin, *Instructor*

and

Robert W. Beal, *Instructor*

Department of Theoretical and Applied Mechanics, Iowa State College

SYNOPSIS

The 1936 reconnaissance pullout bond tests raised important questions, some of which challenged venerable concepts. The comprehensive tests here reported bear out last year's indications that

(a) Bond resistance does not increase in proportion to the increase in compressive strength of concrete.

(b) The total bond resistance of a plain bar increases with added length of embedment up to some limiting length (about 24 diameters for these tests) above which there is little or no gain from added length of embedment.

(c) The unit bond resistance for a plain bar decreases as the length of embedment increases for all lengths of embedment.

(d) Deformed bars slip initially at bond stresses somewhat above those for plain bars and invariably fail ultimately by splitting the concrete.

Among the several more or less new findings are

(e) That pullout specimens can be made to give true indications of bond resistance of similar bars in beams.

(f) That the increase of maximum bond resistance over that at initial slip, for plain bars becomes uniformly less as the stress in the bar increases and at yield point stress there is no appreciable increase in bond above that at initial slip.

(g) The progressive nature of bond failure is shown by measured strains at the compressive concrete surfaces of beam and pullout specimens.

The present tests cover two strengths of concrete (3,000 and 5,000 p.s.i. at 28 days), 3 sizes of plain and deformed high elastic limit steel bars ($\frac{3}{8}$, $\frac{5}{8}$ and $\frac{3}{4}$), several depths of concrete cover over deformed bars, pullout specimens with embedments ranging from 3 to 24 in. (length diameter ratios from 4 to 64), beam-type specimens with steel percentages ranging from 0.552 to 2.76. These gave failures in bond and/or diagonal tension, tension in the steel, and one compressive concrete failure. These data supply a factual basis for determination of factors of safety against each of the several types of failure. Factors of safety against failure by bond and/or diagonal tension are somewhat higher than those against failure by tension (yield point stress) in the steel. Only a limited portion of the strain data secured is presented in this paper.

The tests of Chapter V, Reconnaissance Pullout Series of 1936, (1, p. 82)[1] raised two major questions relative to the bond resistance developed between steel and concrete and refocused attention upon a number of secondary aspects on which existing data or interpretations thereof are not in agreement.

This year's tests, Series of 1937, are a continuation designed to extend and verify or qualify the implications of last year's limited data.

THE MAJOR QUESTIONS

1. *Strength of Concrete.* The 1936 tests indicated that while there is some increase in bond resistance for increased strength of concrete, the increase in bond strength is not in proportion to the increase in compressive strength. Since,

[1] Figures in parentheses refer to list of references at end.

for years, allowable bond stresses have been specified as a fixed percentage of the 28-day ultimate compressive strength, this indication was disquieting Fortunately this provision applied only to ultimate 28-day strengths up to 4,000 lb per sq in, above which no further increase in bond stress was permitted See subsequent discussion on Factors of Safety

2 *Length of Embedment* For the conditions of the 1936 length-of-embedment pullout series, the gain in bond resistance, for increased length of embedment, lacked much of being in direct proportion to the added embedment of the bar For all three strengths of concrete the total pull developed by the plain bars ceased to increase with added length of embedment beyond a length-diameter ratio (L/D) of about 20 This indication was also disturbing because it too branded as on the unsafe side what has, for many years, been an accepted design practice

At least two features of the 1936 tests, however, created uncertainty as to whether or not the findings relative to length of embedment could be accepted as general

(a) The length of embedment tests were nearly all on small one-size bars (¼ in)

(b) At the longer embedments the steel had reached its yield point stress before initial slip occurred There was also recognized the possibility that the slightly decreasing diameter of the bar with increasing stress in the steel may have lowered the unit bond resistance even before yield point stress was attained Certainly at the yield point there is ample basis for suspecting that pronounced progressive letting go along the bar occurs

Obviously last year's indication on length of embedment was vital and demanded further study

OTHER QUESTIONS

3 *Validity of the Pullout Specimen* In a beam the concrete surrounding the steel is in tension and in the pullout specimen it is under mild compression Recognition of this difference has given rise to a question of whether or not a pullout specimen can give a correct indication of the bond resistance developed in a beam The lack of conclusive data has left recognized authorities aligned on opposing sides of this question

There are also other questions such as that of the influence of the usual different casting positions of beams and pullouts on the type of contact secured between steel and concrete Certainly pullout tests are simpler and cheaper to make than beam tests, but their value depends upon the extent to which the pullout test results can be accepted as representative of performance under beam conditions

4 *Permissible Limit of Slippage* In translating pullout test results to the conditions that prevail in a beam, what is the limit of permissible slippage? Since the earlier pullout tests, it has been known that much of the higher pullout bond resistance of deformed bars cannot be utilized in beams because the prior slippage is sufficiently great, already to have produced failure The same question applies to the effectiveness of special anchorage because of the accumulated slip that may develop in the length between point of initial slip and the anchorage

5 *Splitting or Spalling of the Concrete from the Wedging Action of Deformed Bars* The lugs on the deformed bars, after they do become effective, can develop high resistance to slip only if the concrete is reinforced against splitting by spiral reinforcement or by great depth of cover How effective are different practicable depths of cover in increasing resistance to splitting?

6 *Strains in the Concrete* Strains measured along the concrete surfaces of pullout specimens and at the compressive and tensile faces of the concrete beams, should cast important light on the nature of interaction between steel and concrete The 1936 reconnaissance of Ernst and Dunagan into this field (7 p 96) indicated the fundamental nature of the information obtainable

7 *Cracks* The formation and spacing of cracks are always important items regarding the details of which much remains to be determined Accurately measured surface strains supply positive information on crack formation

8 *Factors of Safety of Beam-Type Specimens on the Basis of Current Design Practice* During recent years there has been a growing feeling among engineers that bond is perhaps the most vulnerable of the several stresses against which they design These data on beam-type specimens, tested to their ultimate strength supply important evidence on this aspect of the design

9 *Compressive Stresses in Concrete Beams* The test literature on reinforced concrete beams shows very few failures by compression in the concrete and where these have occurred the percentage of tensile reinforcement has been one and one half to twice that for a balanced design Moreover the computed concrete stress in flexure has been greatly in excess of the compressive strength as measured directly by a prism or cylinder test The answer is not, of course, that beam concrete is stronger than cylinder concrete but simply that, due to redistribution from plastic flow of the compressive concrete most highly stressed, the distribution is parabolic or rectangular rather than straight-line as the flexure formula assumes These tests with steel percentages ranging from 0 552 to 2 761 (Tables 1 and 2, col (a)) supply important supplementary data on this point There was but one compressive beam failure in the entire series (Beam No 11, Table 1, Steel 2 454 per cent, computed compressive strength of concrete 5,220 lb per sq in, actual compressive strength of concrete 3,170 lb per sq in)

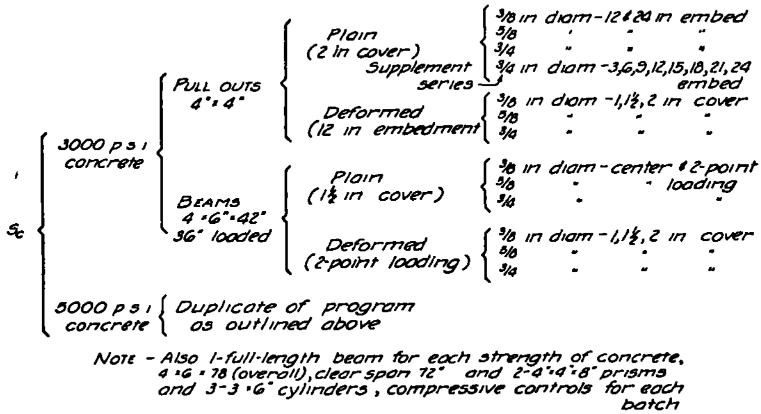

Figure 1 Outline of Program, Indicating Types of Specimens and Variables Studied

OUTLINE OF THE TESTS

Summary of Variables, Specimens and Batches Figure 1 shows in diagrammatic form an outline of the tests The variables are seen to be

2 strengths of concrete (3,000 and 5,000 lb per sq in)

2 primary types of bond specimen (pullout and half beam)
1 supplementary type of bond specimen (one full-length beam for each strength of mixture)
2 loadings for plain-bar half beams (center and 2 point) (All deformed bar beams had 2 point loading only)
2 types of high elastic limit steel bars (plain and deformed)
3 sizes of each type of bar (⅜, ⅝ and ¾ in round)
3 depths of concrete cover (1, 1 5 and 2 in from centers of bars) over

3–3 by 6 in cylinders (standard compressive specimens)

Summary of Specimens

Half or simulated beams	30
Full length beams	2
4 by 4 by 12 in pullout specimens, plain bars (main series)	6
4 by 4 by 24 in pullout specimens, plain bars (main series)	6
4 by 4 by 12 in pullout specimens, def bars (main series)	18
4 by 4 (3 to 24 in) pullout specimens, plain bars (sup series)	16
4 by 4 by 8 in compressive prisms	38
3 by 6 in compressive cylinders	57
Total specimens tested	173

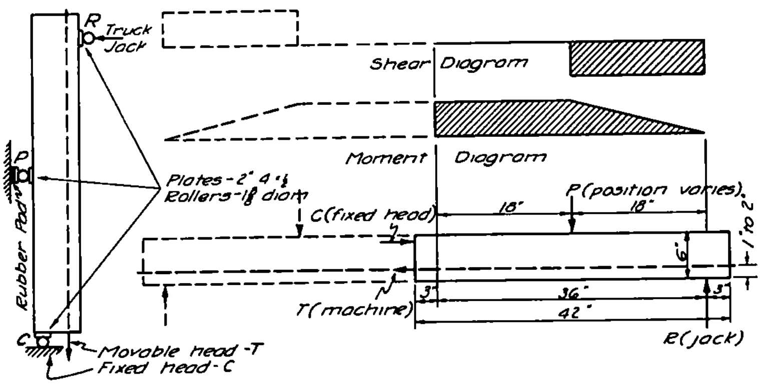

Figure 2 The Beam-Type Specimen and Manner of Loading It

the deformed bars only (but for both the half beams and the pullout specimens)
2 lengths of embedment for plain-bar pullout specimens of main series (12 and 24 in)
1 length of embedment for the deformed bar pullout specimens (12 in)
8 lengths of embedment for the ¾ in plain bars of the supplementary pullout series (3, 6, 9, 12, 15, 18, 21 and 24 in)

Compressive control specimens were cast from each of the 19 batches of concrete as follows

2–4 by 4 by 8 in prisms (analogous to pullouts)

Selection of Types of Primary Specimens Because of the unsettled differences of opinion and apparent conflict of evidence on whether or not the pullout specimen supplies correct indications on relative bond resistance as applied to steel in beams (1, p 92), a simulated beam or half-beam specimen was evolved as shown in Figure 2[2]

The analogy to the beam should be apparent A full series of parallel pull-

[2] This arrangement was sketched up several years ago by Dr Glenn Murphy of this department, while a graduate student at the University of Illinois It is similar in some respects to one used by Posey in his tests on hooks (1, p 95) More recently Professor M O Withey mentioned, in conversation, the possibility of using a similar device

out tests was conducted and, in addition, two full length beams were cast, vertically,[3] one of each mixture, and tested horizontally, as beams, as a final cross-tie from half-beam to beam

Materials and Mixtures The rail steel bars were all from a single shipment furnished by the Laclede Steel Company of St Louis In every instance the tensile strength test was performed on a piece cut from the same bar that went into the specimen Moreover pull-out specimens carry the same numbers as the companion half-beam specimens which were cast from the same batch of concrete, and are reinforced from the same bar of steel

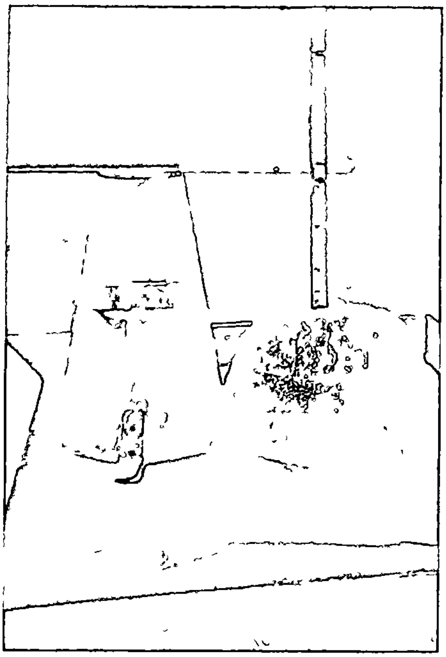

Figure 3 Slump Test on a Typical Batch of the Concrete

The cement was a single lot of Hawkeye standard portland and was screened over a No 14 sieve, thoroughly premixed and stored in tight paper-lined cans

The Des Moines river sand used was all from a single load of the usual 0-4 grading but was somewhat lacking in fines from having been over-washed The coarse aggregate was Des Moines river gravel $\frac{1}{4}$-$\frac{3}{4}$ in and all from a single load

The proportions of the 3,000 lb per sq in (28 day) concrete were 1 3 83 1 67 with a water-cement ratio of 0 675, all by weight Unit weight as cast was 143 lb per cu ft and Young's modulus was approximately 4,000,000 lb per sq in

The 5,000 lb per sq in proportions were 1 1 85 1 55 with a water-cement ratio of 0 425 The unit weight as placed was 148 lb per cu ft, and Young's modulus ranged slightly under 5,000,000 lb per sq in Both concretes were designed for a 6-in slump which was maintained in all batches within an inch either way The mixtures were somewhat over sanded, fat and workable as shown by Figure 3 (5,000 lb per sq in mixture)

All specimens were cast vertically in steel forms as shown in Figures 4 and 5[4] Figure 6 shows the specimens from a single batch of plain-bar specimens The bar in a pullout specimen is a part

[3] A vertical casting was used for the two full-length beams in order to maintain similarity with the half-beams, as well as with the pullout specimens The two full-length beams were introduced to check the validity of the beam-type or half-beam specimen and it was deemed better not to complicate the results by the introduction of that other important variable "the horizontal or vertical position as cast" The importance of casting orientation has received some attention as, for example, Slater, Richart, and Scofield in Reference 5 of the 1936 report (1, p 94) To secure useful quantitative data on that variable with all its manifold ramifications would require a carefully planned and executed investigation of no mean proportions Further work along those lines is needed, however

[4] These were furnished by W A Jennings of Des Moines and proved to be very satisfactory for casting the varied lengths and sizes of specimens

of the same rod as that in the half-beam that carries the same specimen number.

Forms were stripped at 24 hours and all specimens were cured under water for 27 additional days. All specimens were tested at 28 days within a few hours after removal from water.

Testing and Instrumentation: Compressive 3 by 6 in. control cylinders were tested in a 75,000 lb. capacity Baldwin-Southwark cement testing machine and the 4 by 4 by 8 in. compressive prisms were tested in a 100,000 lb. Olsen screw gear-lever machine. Longitudinal stress-strain data were secured with compressometers similar to those described and pictured in (2, p. 467 and 469).

Pullout specimens were tested in a 20,000 lb. hand-operated Olsen screw-gear lever testing machine. The bearing end was set in plaster of paris, the self aligning spherical bearing block not be-

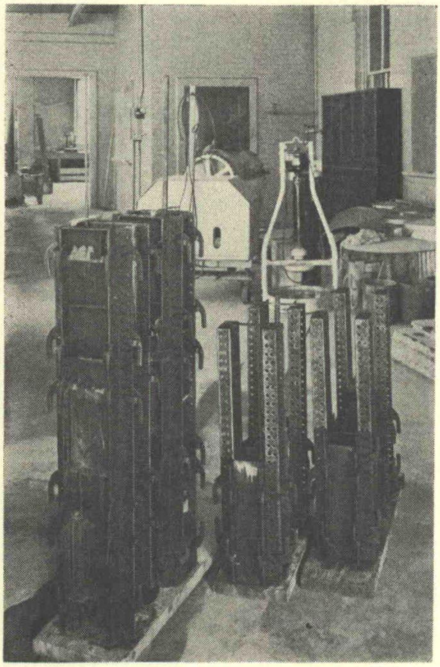

Figure 4. Steel Forms for Beam Type and Pull-Out Specimens

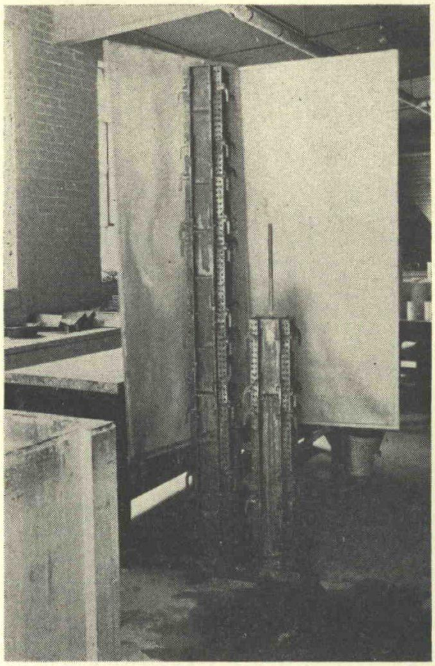

Figure 5. One Full-Length Beam and a Half-Length Beam-Type Specimen in Steel Forms.

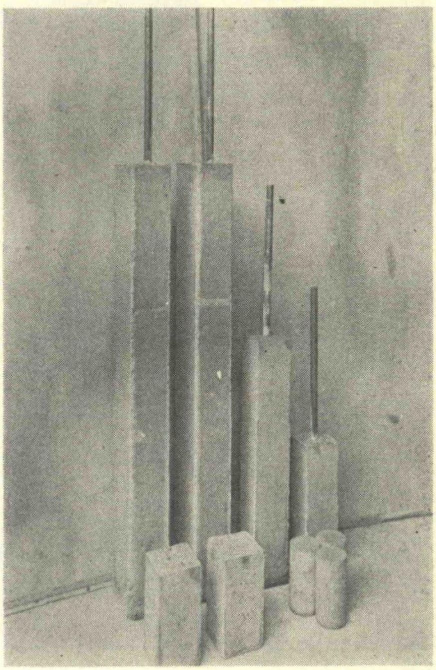

Figure 6. Specimens from One Batch of Concrete

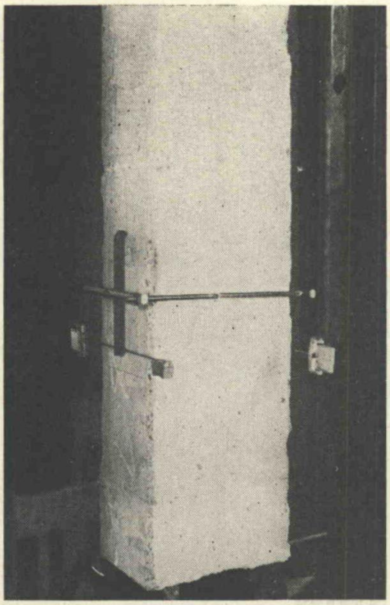

Figure 7. A Pair of Marten's Mirror Extensometers on Opposite Faces of a Specimen.

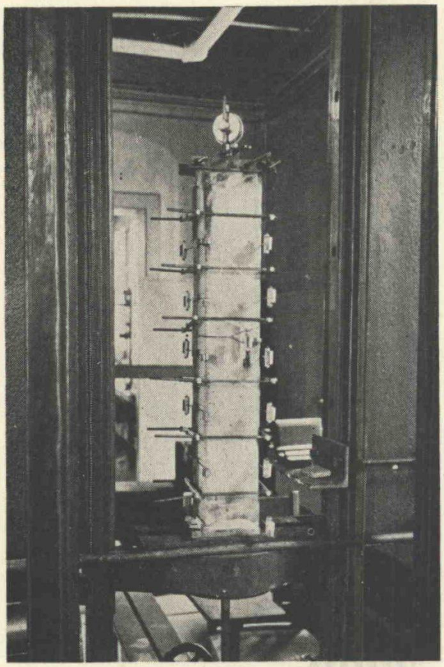

Figure 8. A 24-inch Pull-Out Specimen Ready for Test

ing sufficiently rigid for use with the sensitive Marten's mirror strainometers used on concrete surfaces.

Thirteen Marten's mirrors were placed on each 24-in., and 7 mirrors on each 12-in pullout specimen (Figures 7 and 8) after the method of Ernst and Dunagan (7, p. 96-99) which was an extension of methods earlier used in other kinds of

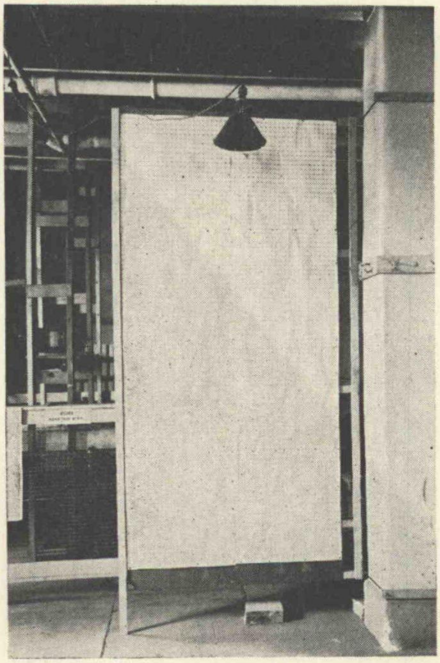

Figure 9. Grid Screen for Use with Marten's Mirrors.

tests by A. N. Johnson (3, p. 1025), and others.

Twenty-one mirrors were used on each beam-type specimen. The Marten's mirror extensometers were mounted in pairs on opposite sides of the member (which, for the beams, were the tensile and compressive faces) except when some obstruction prevented. Strains in the concrete were indicated on successive 4 inch gage lengths, which overlapped $\frac{1}{4}$ in. At intervals on one side of each beam-type specimen, at or near its neutral surface, about five fixed correction mirrors (A in Figure 10) were also mounted to obtain readings for corrections due to the rotations caused by bending as the test progressed. Usually such corrections have been made automatically by mounting two mirrors rotating in opposite directions on opposite gage lines, the recorded reading being the mean of the two. Here there was but one mirror per gage line and the effect of rotation was corrected by taking the reading on the screen as the difference between the actual reading and the change in reading on the nearest fixed mirror.

It was important that the bottom of the beam, where the bar entered, be kept free, both from lateral movement and from rotation, for otherwise there would be cramping and spalling of the concrete around the bar such as did occur frequently just as the beam failed, as is illustrated at the bottom of Figure 15. This necessitated provision for rotation and movement at the upper end of the beam to allow for the progressive bending under test.

The upper end reaction of the beam (Figure 2 and 11), was carried by a U frame through a ball-bearing truck jack at R, by which the upper end could be made to rotate sufficiently to maintain a constant reading on a fixed mirror located at the neutral surface near the bottom of the beam. This controlled the rotation. The lateral deflection between the bottom bearing plate C, over the bottom roller, and the load-point P was controlled closely by inserting a rubber pad, of suitable thickness (as determined by trial), behind the bearing plate at P. A 1/1000 in. dial indicator near the bottom indicated only a slight lateral movement at the base and no difficulty from premature bottom spalling was encountered.

As shown in Figure 11, each of the standards of the testing machine was reinforced by bolting a 7-inch I-beam along it, to supply the necessary added lateral strength and rigidity. The load point at P could be shifted along the beam as desired to simulate either center loading or any desired two point symmetrical loading on the equivalent full length beam. The top 1/10,000 in. dial indicator for detecting and measuring end slip, and the top and bottom 1/1000 in. horizontal dial indicators for deflections are shown in Figure 11.

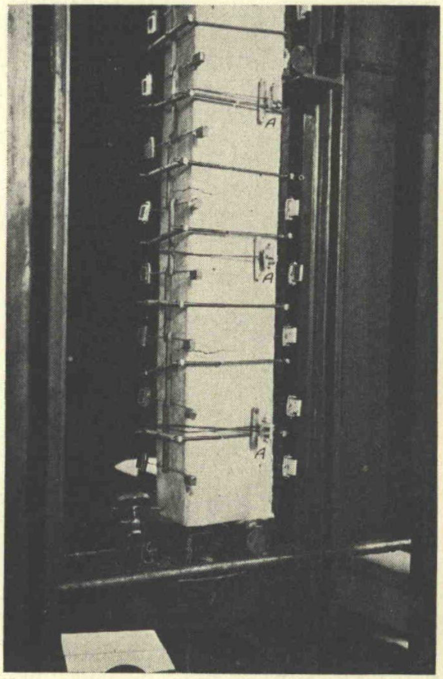

Figure 10. Close Up of Mirrors on a Beam-Type Specimen that Failed in Tension. Marten's mirror Extensometers Located on Tensile and Compressive Faces. Three of the Fixed Correction Mirrors "A" Show Along the Side of the Member.

On Figure 8 is shown one fixed mirror for checking and correcting, if necessary, on a pullout specimen. For the pullouts with variable cover, there was some bending from eccentricity and two or more fixed mirrors were used at intervals along the side.

Figure 12 shows the mirrors of a beam-type specimen as viewed from the screen. One of the two full-length horizontal beams is shown ready for test with dials and mirrors in place on one-half the beam in Figure 13 and an end view of the same looking along the beam toward the grid screen is shown as Figure 14. The grid shows dimly as does the small hole for the transit telescope. To arrange this beam with all mirrors visible from the same position required care and ingenuity.

The grid screen of Figure 9 was larger than necessary for the range of readings but the added size was a convenience in setting up and adjusting the mirrors. After the testing routine was well established, it became possible to set up and run a complete test on a beam-type specimen in three hours or less.

A check curve and sample calculation for mirror extensometer No. 12 is shown on Figure 19. This is a check rather than a calibration since the mirror was more sensitive than was the 1/10,000 Federal dial used with the Gilkey-Vogt universal calibrator (4, p. 722). One-tenth inch on the screen was the equivalent of about 350 lb. per sq. in. in the steel and about 46 and 56 lb. per sq. in. in the concretes.

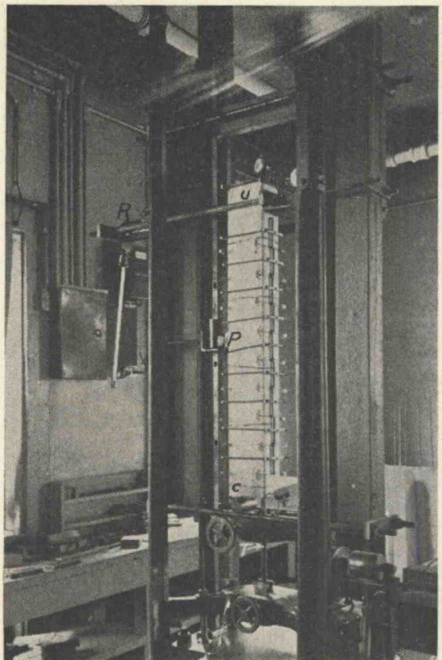

Figure 11. A Beam-Type Specimen Ready for Test. C = Compressive Plate; T = Tensile Reinforcing Bar; P = Load Point where a Rubber Pad Was Inserted Back of the Roller (See Text) and Bearing Plate; U = U Frame for End Reaction; R = Truck Jack for Adjusting Through the U Frame Against Rotation at Bottom as Beam Deflects. See Figure 2, also.

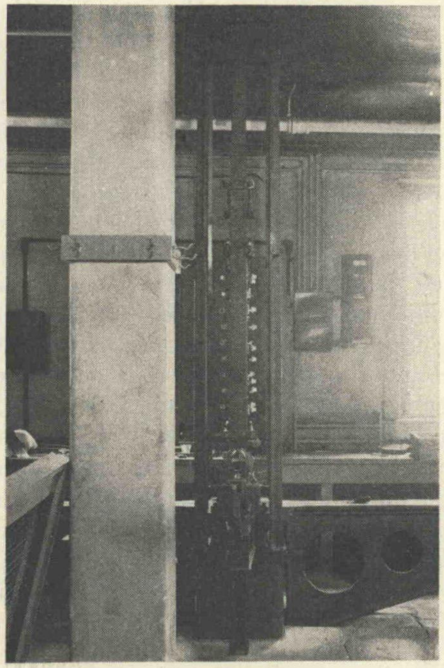

Figure 12. Mirrors on a Beam-Type Specimen as Viewed from the Transit, Located at the Grid Screen.

Figures 15, 16 and 17 show typical beams after failure. Beam No. 26, (Figure 16) had no end slip and failed by tension in the steel. Beams 17 and 31 both failed either in bond or diagonal tension. To separate the two types of failure is usually difficult because con-

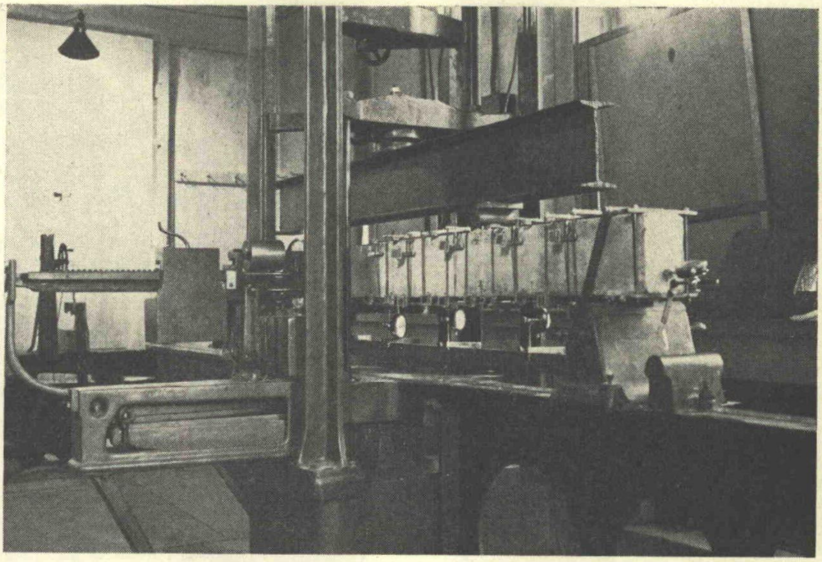

Figure 13. Full-Length Beam No. 31 Ready for Test with Mirrors and Dials Attached

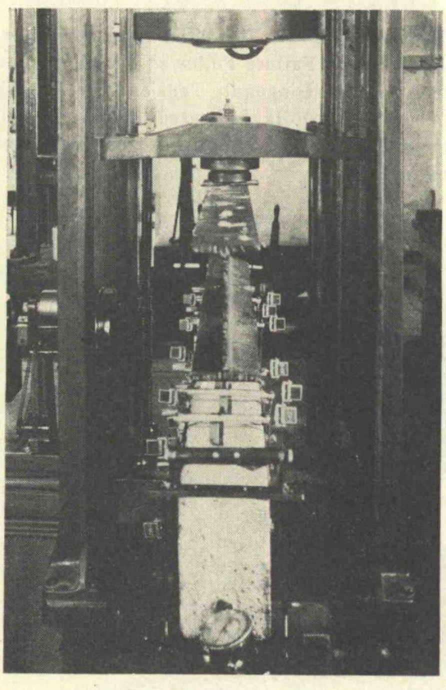

Figure 14. End View of Full-Length Beam No. 31 Looking Toward the Grid Screen. Transit Telescope Projected Through the Hole in the Screen.

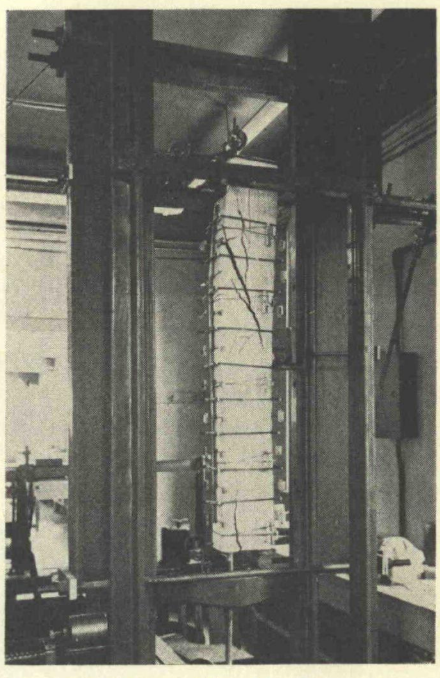

Figure 15. Beam No. 17 After Test. A Typical bond and/or diagonal tension failure. The Secondary Split at the Bottom Was the Result of Lateral Rebound from the Let-Go as Failure Occurred.

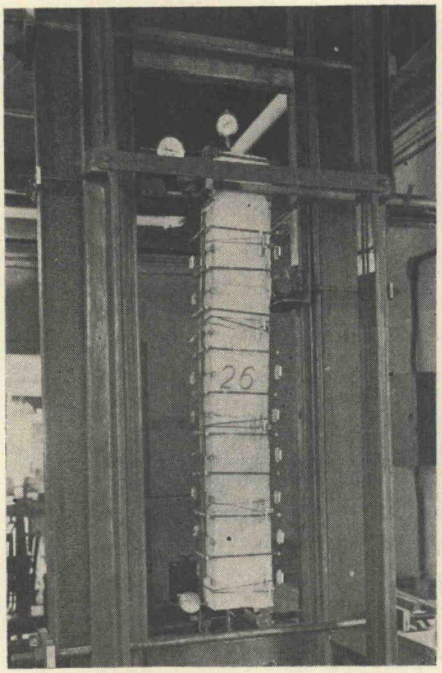

Figure 16. Beam No. 26 After Test. A Typical Failure by Tension in the Steel. Such a Failure Occurred Gradually and There Was No Bottom Split.

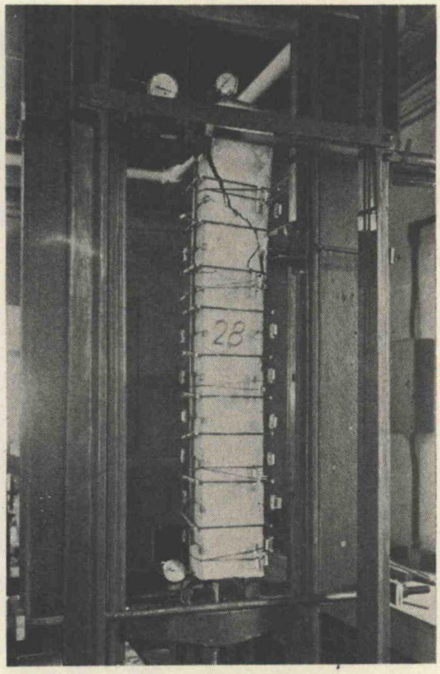

Figure 17. Beam No. 28 After Test. A primary Tension Failure Followed by a Secondary Bond and/or Diagonal Tension Break. The Tension Failure Is Indicated by the Tensile Cracks and the Absence of a Bottom Split from Sudden Recoil.

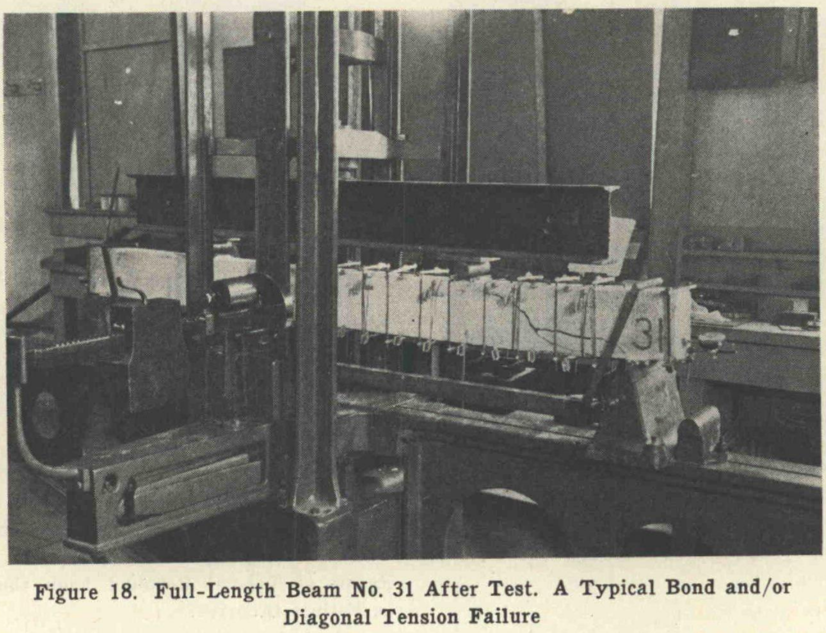

Figure 18. Full-Length Beam No. 31 After Test. A Typical Bond and/or Diagonal Tension Failure

ceivably either one may have been primary Probably most diagonal tension failures were initially failures in bond, however

Loads were applied to both beam and pullout specimens in increments of 1,000 or 2,000 lb , depending upon the maximum load expected A full set of mirror and dial readings was taken for each load increment until the maximum load was reached The load at first end slip

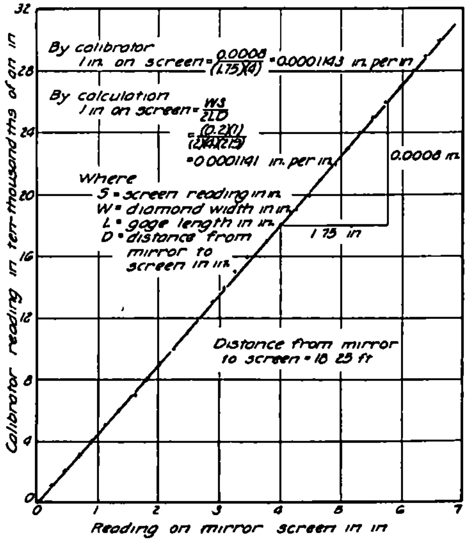

Figure 19 A Check Comparison of Mirror Extensometer No 12 with a 0.0001 in Federal Dial by Means of a Gilkey-Vogt Universal Calibrator. Gage Length 4 in

of bar was noted as was the drag load on the pullouts for plain bars after about 0 08 in slip had brought the frictional resistance almost to a uniform value The nature of a flexural bond failure is such that drag load observations would be meaningless Drag loads were not taken on deformed bar pullout specimens because the load continues to increase until splitting occurs

RESULTS

Table 1 supplies the primary data on the main series for the 3,000 lb per. sq in strength concrete and Table 2 gives the corresponding data for the 5,000 lb per sq in strength concrete Table 3 supplies data on the supplementary series of pullout specimens for $\frac{3}{4}$-in plain bars with variable length of embedment

The stress-strain characteristics of the concrete, standard cured at 28 days, are shown on Figure 20 The strengths and values of Young's modulus for the dif-

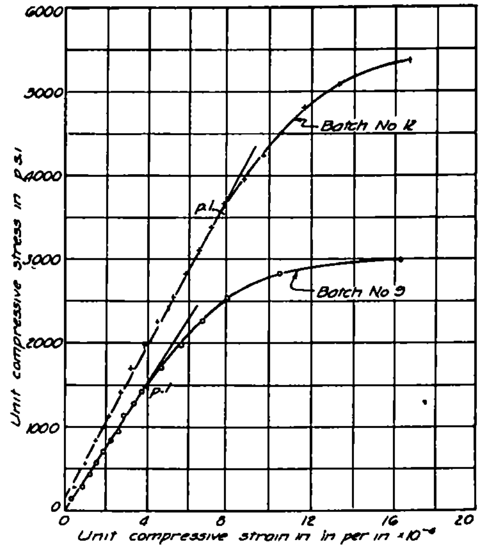

Figure 20 Typical Stress-Strain Curves for 3 by 6 in Compressive Control Cylinders

ferent batches are given in Column (c) of Tables 1 and 2

Overall Comparisons at First Slip The data from the length of embedment pullout series of 1936 (1, p 82) arranged in the order of unit bond stress at first slip are shown on Figure 21 The specimens are identified fully by the code The unit bond stresses corresponding to the yield point stress in the steel, the ultimate unit bond stress and the residual or drag bond stress after sliding 0 05 in are also plotted on the figure These include three strengths of concrete and four lengths of embedment for $\frac{1}{2}$-in bars, mainly There are a few specimens with $\frac{3}{8}$-in bars.

TABLE 1

Test Results for Main Series 3000 Lb (Nominal Strength) Concrete

Spec No	Bar: Size / Type / Per cent for beam	Properties of concrete and steel, lb per sq in: Yield point of bar (lb) and (lb per sq in)	Properties of concrete and steel: Compr str of conc / Mod elast $\times 10^{-6}$	Pullout tests: Cover (in)	Pullout tests: Length of embedment (in)	Pullout tests (First slip / Ultimate): Pull on bar (lb)	Pullout tests (First slip / Ultimate): Bond strs (lb per sq in)	Tests of beam-type specimens (values at first slip above) (values at ultimate below): Cover (in)	Load[b] point	Pull on bar as tested (lb)	Total load beam equiv (lb)	Slip of bar at ult (in $\times 10^{-4}$)	Type of failure	$f_s{}^c = \frac{Load}{A_s}$ (Lb per sq in)	$f_c = \frac{2M}{jkbd^2}$ (Lb per sq in)	$u = \frac{V}{\Sigma o jd}$ (Lb per sq in)	$u = \frac{Load}{\Sigma o l}$ (Lb per sq in)	$v = \frac{V}{bjd}$ (Lb per sq in)	Computed maximum design loads (lb): Steel ten $20{,}000 = \frac{M}{A_s jd}$	Conc comp $0.4f'_c = \frac{2M}{jkbd^2}$	Bond $\left.\begin{matrix}0.04f'_c\\0.05f'_c\end{matrix}\right\} = \frac{V}{\Sigma o jd}$	Diag ten $.02f'_c = \frac{V}{bjd}$	Factor of safety (first slip above) (ultimate below): Steel ten	Conc comp	Bond	Diag ten	Spec No
Col.	a	b	c	d	e	f	g	h	i	j	k	l	m	n	o	p	q	r	s	t	u	v	w	x	y	z	Col
1	3/8 p	6800	3235	2	12	6660	471	1½	L/4	7800	3250	73	Ten	70600	3000	337	316	99	*1010*	1404	1250	2120	3 22				1
	0 613	61500	4 09			6810	481			8000	3330			72400	3070	345	324	101					3 30				
3	3/8 p	6960	3235	2	24	7035	249	1½	L/2	7760	1624	24	Ten	70200	3000	168	169	50	*505*	702	1250	2120	3 22				3
	0 613	62900	4 09			7035	249			7760	1624			70200	3000	168	169	50					3 22				
5	5/8 p	19400	3360	2	12	12910	548	1½	L/4	20650	8120	22	B&DT	67300	5240	528	500	259	2670	2080	*2065*	*2100*			3 87	3 93	5
	1 704	63200	4 22			14330	608			20650	8120			67300	5240	528	500	259							3 87	3 93	
7	5/8 p	19560	3360	2	24	17000	361	1½	L/2	20350	4260	13	Ten	66400	5510	278	265	136	*1335*	1040	2065	2100	3 19				7
	1 704	63700	4 22			19000	402			20350	4260			66400	5510	278	265	136					3 19				
9	3/4 p	27000	3170	2	12	15300	541	1½	L/4	20000	7500	52	B&DT	45400	4350	414	404	246	3750	2180	*2280*	*1940*			3 29	3 87	9
	2 454	61100	4 05			17270	610			21000	7880			47600	4570	437	424	258							3 45	4 06	
31[a]	3/4 p	26750	2880					1½	L/4		6000	Spun	B&DT	32000	3480	333	286	196	3750	1980	*2070*	*1760*			2 91	3 41	31[a]
	2 454	60500	3 85								6000			32000	3480	333	286	196							2 91	3 41	
11	3/4 p	27000	3170	2	24	22920	405	1½	L/2	NS[d]	NS	NS	Comp	NS	NS	NS	NS	NS	1875	*1090*	2280	1940		NS			11
	2 454	61100	4 05			24420	432			24000	4490			54400	5220	250	261	148						4 12			
19	3/8 d	6720	3130	1	12	7040	498	1	L/4	NS	NS	NS	Ten	NS	NS	NS	NS	NS	*1130*	1545	1695	2310	NS				19
	0 552	60800	3 95			7100	502			8000	3440			72400	2770	318	324	93					3 04				
13	3/8 d	7810	3350	1½	12	8000	566	1½	L/4	NS	NS	NS	Ten	NS	NS	NS	NS	NS	*1010*	1455	1620	2200	NS				13
	0 613	70600	4 20			8915	630[e]			9000	3380			81400	3120	350	364	103					3 35				
25	3/8 d	7660	3180	2	12	8000	566	2	L/4	NS	NS	NS	Ten	NS	NS	NS	NS	NS	*892*	1140	1360	1845	NS				25
	0 690	69300	4 10			8430	595			10460	3340			94600	3730	391	423	115					3 74				
21	5/8 d	20780	3085	1	12	12000	509	1	L/4	14000	6020	65	B&DT	45700	3260	351	339	172	3000	2275	*2650*	*2160*			2 27	2 79	21
	1 534	67700	3 88			14570	617[e]			14000	6020			45700	3260	351	339	172							2 27	2 79	

TABLE 1—*Continued*

TESTS RESULTS FOR MAIN SERIES, 3000 LB (NOMINAL STRENGTH) CONCRETE

Spec No	Bar: Size, Type, Per cent for beam	Properties of concrete and steel lb per sq in: Yield point of bar (lb) and (lb per sq in)	Properties: Compr str of conc / Mod elast ×10⁻⁶	Pullout tests: Cover (in)	Pullout tests: Length of embedment (in)	Pullout tests (first slip / ultimate): Pull on bar (lb)	Pullout tests (first slip / ultimate): Bond strs (lb per sq in)	Beam-type: Cover (in)	Beam-type: Load[b] point	Beam-type: Pull on bar as tested (lb)	Beam-type: Total load beam equiv (lb)	Beam-type: Slip of bar at ult (in ×10⁻⁴)	Beam-type: Type of failure	$f_s' = \frac{Load}{A_s}$ (Lb per sq in)	$f_c = \frac{2M}{jkbd^2}$ (Lb per sq in)	$u = \frac{V}{\Sigma ojd}$ (Lb per sq in)	$u = \frac{Load}{\Sigma ol}$ (Lb per sq in)	$v = \frac{V}{bjd}$ (Lb per sq in)	Computed max design loads (lb): Steel ten 20,000 = $\frac{M}{A_s jd}$	Conc comp 0 4f_c' = $\frac{2M}{jkbd^2}$	Bond $\left.\begin{matrix}0\ 04f_c'\\0\ 05f_c'\end{matrix}\right\} = \frac{V}{\Sigma ojd}$	Diag ten 02f_c' = $\frac{V}{bjd}$	Factor of safety (first slip above) (ultimate below): Steel ten	Conc comp	Bond	Diag ten	Spec No
Col	a	b	c	d	e	f	g	h	i	j	k	l	m	n	o	p	q	r	s	t	u	v	w	x	y	z	Col
15	5/8 d	21170	3350	1½	12	16000	679	1½	L/4	16000	6000	10	B&DT	52200	3880	391	388	192	2670	2070	*2570*	*2095*			2 33	2 86	15
	1 704	68900	4 20			20900	885[e]			16000	6000			52200	3880	391	388	192							2 33	2 86	
27	5/8 d	15350	3180	2	12	13660	580	2	L/4	NS	NS	NS	Ten	NS	NS	NS	NS	NS	*2350*	1610	2155	1755	NS				27
	1 918	50000	4 10			19100	809[e]			18000	5740			58700	4530	423	436	208					2 45				
23	3/4 d	23190	3085	1	12	14000	495	1	L/4	19640	8450	100	B&DT	44500	4090	419	397	247	4200	2550	*3110*	*2110*			2 71	4 00	23
	2 209	52400	3 88			14000	495[e]			19640	8450			44500	4090	419	397	247							2 71	4 00	
17	3/4 d	23000	3130	1½	12	12000	424	1½	L/4	19240	7200	60	B&DT	43400	4170	400	388	235	3750	2150	*2820*	*1915*			2 55	3 75	17
	2 454	52100	3 95			17480	618[e]			19880	7440			45000	4310	413	401	243							2 64	3 89	
29	3/4 d	23660	3040	2	12	16000	566	2	L/4	22700	7240	55	B&DT	51400	5140	456	458	268	3300	1710	*2410*	*1640*			3 01	4 41	29
	2 761	53600	3 99			21000	742[e]			23430	7480			53100	5310	471	473	277							3 11	4 56	

[a] Full length beam, tested horizontally [b] As referred to a conventional full-length beam [c] Except beam No 31 where $f_s = M/A_s jd$
[d] NS = No end slip of bar [e] Failure was by splitting of concrete Concrete proportions, 1 3 83 1 67 w/c = 0 675 (all by weight)
Slump = 6 in ± 1 Wt per cu ft = 143 lb

TABLE 2

TEST RESULTS FOR MAIN SERIES 5000 LB (NOMINAL STRENGTH) CONCRETE

	Bar	Properties of concrete and steel lb per sq in		Pullout tests				Tests of beam-type specimens (values at first slip above) (values at ultimate below)											Computed maximum design loads (lb)				Factor of safety (first slip above) (ultimate below)				
	Size / Type					First slip / Ultimate								Lb per sq in													
Spec No	Per cent for beam	Yield point of bar (lb) and (lb per sq in)	Compr str of conc / Mod elast ×10-6	Cover (in)	Length of embedment (in)	Pull on bar (lb)	Bond strs (lb per sq in)	Cover (in)	Load[b] point	Pull on bar as tested (lb)	Total load beam equiv (lb)	Slip of bar at ult (in ×10-4)	Type of failure	$f_s' = \frac{\text{Load}}{A_s}$	$f_c = \frac{2M}{jkbd^2}$	$u = \frac{V}{\Sigma ojd}$	$u = \frac{\text{Load}}{\Sigma ol}$	$v = \frac{V}{bjd}$	Steel ten $20{,}000 = \frac{M}{A_sjd}$	Conc comp $0.4f_c' = \frac{2M}{jkbd^2}$	Bond $\left.\begin{matrix}160\\200\end{matrix}\right\} = \frac{V}{\Sigma ojd}$	Diag ten $.02f_c' = \frac{V}{bjd}$	Steel ten	Conc comp	Bond	Diag ten	Spec No
Col	a	b	c	d	e	f	g	h	i	j	k	l	m	n	o	p	q	r	s	t	u	v	w	x	y	z	Col
2	3/8 p	7230	5450	2	12	7000	495	1½	L/6	6800	3970	115	Ten	61600	2690	416	385	121	*1525*	3220	1560	3610	2 60				2
	0 613	65400	4 96			7410	524			8000	4670			72500	3160	479	452	141					3 06				
4	3/8 p	6920	5450	2	24	7540	267	1½	L/2	7200	1400	55	Ten	65200	2850	144	157	42	*508*	1073	1560	3610	2 76				4
	0 613	62600	4 96			7705	273			8000	1560			72500	3170	160	174	47					3 07				
6	5/8 p	15410	5020	2	12	15200	645	1½	L/2	16900	3290	96	Ten	55100	4590	211	220	104	*1350*	1440	2490	3180	2 43				6
	1 704	50200	4 90			15700	665			17400	3390			56700	4720	217	227	107					2 51				
8	5/8 p	16333	5020	2	24	17100	363	1½	L/6	18550	10800	68	Ten	60400	5030	696	630	342	*4050*	4320	2490	3180	2 67				8
	1 704	53200	4 90			17300	367			18860	11000			61400	5110	707	640	347					2 71				
10	3/4 p	25630	5100	2	12	16000	566	1½	L/2	NS[d]	NS	NS	Ten	NS	NS	NS	NS	NS	*1904*	1645	2930	3170	NS				10
	2 454	57900	4 87			19630	695			29160	5670			66000	7040	310	317	183					2 98				
12	3/4 p	27000	5100	2	24	28640	507	1½	L/6	18670	10880	86	B&DT	42300	4500	595	529	350	5712	4935	*2930*	*3170*			3 71	3 43	12
	2 454	61100	4 87			30000	530			20000	11670			45300	4820	638	567	375							3 98	3 68	
32[a]	3/4 p	25800	5438					1½	L/6		9600	5	B&DT	33600	3970	525	420	309	5712	5260	*2930*	*3380*			3 27	2 84	32[a]
	2 454	58300	4 93								9600			33600	3970	525	420	309							3 27	2 84	
20	3/8 d	7540	4892	1	12	8550	604	1	L/6	NS	NS	NS	Ten	NS	NS	NS	NS	NS	*1700*	3430	2180	3620	NS				20
	0 552	68200	4 67			9925	702[f]			9160	6110			83000	3480	561	518	165					3 60				
14	3/8 d	6940	5435	1½	12	8740	618	1½	L/6	NS	NS	NS	Ten	NS	NS	NS	NS	NS	*1525*	3210	1950	3600	NS				14
	0 613	62800	4 79			9905	701			7540	4400			68300	2970	451	426	133					2 89				
26	3/8 d	6930	5332	2	12	10130	717	2	L/8	NS	NS	NS	Ten	NS	NS	NS	NS	NS	*1800*	3480	1730	3130	NS				26
	0 690	62700	4 92			10630	753			8300	5530			75100	3390	753	587	188					3 07				
22	5/8 d	15960	4960	1	12	14000	594	1	L/6	15500	10300	119	Ten[e]	50600	4030	595	525	293	4520	5090	*3480*	*3510*			2 96	2 94	22
	1 534	52000	4 86			16880	715[f]			17860	11900			58200	4640	685	605	337							3 42	3 39	

TABLE 2—*Continued*

TEST RESULTS FOR MAIN SERIES, 5000 LB (NOMINAL STRENGTH) CONCRETE

Spec No	Bar: Size / Type / Per cent for beam	Properties of concrete and steel lb per sq in: Yield point of bar (lb) and (lb per sq in)	Properties: Compr str of conc / Mod elast ×10⁻⁶	Pullout tests: Cover (in)	Pullout tests: Length of embedment (in)	Pullout tests (First slip / Ultimate): Pull on bar (lb)	Pullout tests (First slip / Ultimate): Bond strs (lb per sq in)	Beam-type: Cover (in)	Beam-type: Load[b] point	Beam-type: Pull on bar as tested (lb)	Beam-type: Total load beam equiv (lb)	Beam-type: Slip of bar at ult (in ×10⁻⁴)	Beam-type: Type of failure	$f_s{}^c = \frac{Load}{A_s}$ (Lb per sq in)	$f_c = \frac{2M}{jkbd^2}$ (Lb per sq in)	$u = \frac{V}{\Sigma o jd}$ (Lb per sq in)	$u = \frac{Load}{\Sigma o l}$ (Lb per sq in)	$v = \frac{V}{bjd}$ (Lb per sq in)	Steel ten 20,000 = $\frac{M}{A_s jd}$	Conc comp 0 4f_c' = $\frac{2M}{jkbd^2}$	Bond $\left.\begin{matrix}160\\200\end{matrix}\right\}$ = $\frac{V}{\Sigma o jd}$	Diag ten 02f_c' = $\frac{V}{bjd}$	Steel ten	Conc comp	Bond	Diag ten	Spec No
Col	a	b	c	d	e	f	g	h	i	j	k	l	m	n	o	p	q	r	s	t	u	v	w	x	y	z	Col
16	5/8 d	16950	5435	1½	12	18160	770	1½	L/6	NS	NS	NS	Ten	NS	NS	NS	NS	NS	*4050*	4670	3110	3440	NS				16
	1 704	55200	4 79			24380	1032[f]			19000	11080			61900	5140	712	644	349					2 73				
28	5/8 d	21370	5332	2	12	19740	838	2	L/8	17580	11700	74	Ten[e]	57300	4970	854	621	420	*4760*	5030	2740	2980	2 46				28
	1 918	69600	4 92			30000	1270[f]			23150	15440			75500	6550	1125	818	553					3 25				
24	3/4 d	25110	4960	1	12	16000	566	1	L/6	18720	12500	121	B&DT	42300	4320	613	530	360	6380	5730	*4080*	*3440*			3 06	3 63	24
	2 209	56800	4 86			17780	628[f]			20700	13800			46800	4780	677	586	398							3 38	4 01	
18	3/4 d	24450	4892	1½	12	16000	566	1½	L/6	21000	12250	229	B&DT	47500	5060	670	595	395	5712	4710	*3650*	*3020*			3 35	4 06	18
	2 454	55300	4 67			24000	849[f]			26700	15580			60400	6440	851	756	502							4 27	5 16	
30	3/4 d	23660	5438	2	12	18000	636	2	L/8	25000	16700	20	Ten	56600	6300	1032	885	610	*6720*	5750	3220	2980	2 49				30
	2 761	53600	4 93			30000	1060[f]			25380	16900			57400	6390	1049	897	618					2 51				

[a] Full length beam, tested horizontally [b] As referred to a conventional full-length beam [c] Except beam No 32 where $f_s = M/A_s jd$.
[d] NS = No end slip of bar [e] No 22 and 28 tension primary, B and D T secondary [f] Failure was by splitting of concrete
Concrete proportions, 1 1 85 1 55 w/c = 0 425 (all by weight) Slump = 6 in ± 1 Wt per cu ft = 148 lb

Similar data for all of the plain bar specimens of both of the 1937 series are plotted on Figure 22, the beams of the main series being at the right In like manner Figure 23 gives data on all the 1937 deformed bar specimens Both strengths of concrete and all sizes of bar and length of embedment are included

Bond vs Strength of Concrete Unit bond stresses and ratios of bond resistances to strengths of concrete are shown for first slip and ultimate loads on Figure 24 This diagram includes both the plain and deformed bars and both the pullouts and the beams The steel in many of the specimens had

TABLE 3

TEST RESULTS FOR SUPPLEMENTARY PULLOUT SERIES
All bars plain, ¾ in diameter

Spec No	Length of embedment (in.)	Load on bar in lb (above) Per cent of Y P load (below)				Bond stress in lb per sq in (above) Percent first slip (below)			Slip at max load (in.)
		Yield point	First slip	Ult	Drag at .08 in slip	First slip	Ult	Drag at .08 in slip	
Col	a	b	c	d	e	f	g	h	i
Ultimate Compressive Strength of Concrete 3025 lb per sq in									
3- 3	2.88	25100	3305	4185	2865	488	618	423	0.0100
			13.2	16.7	11.5		126.5	86.5	
3- 6	6.25	25100	6040	8420	5645	411	572	383	0.0064
			24.0	33.5	22.5		139.2	93.2	
3- 9	9.13	25100	8000	9280	5790	372	431	264	0.0035
			31.8	36.9	23.1		115.7	71.0	
3-12	11.75	25100	10555	12065	8300	381	436	300	0.0058
			42.1	48.0	33.1		114.4	78.8	
3-15	15.25	25350	13940	15315	10885	388	427	303	0.0058
			55.1	60.6	43.0		110.0	78.1	
3-18	18.25	25350	19200	21115	11400	447	491	265	0.0058
			75.8	83.5	45.1		109.8	59.3	
3-21	21.25	25350	19725	21585	12700	394	431	254	0.0059
			78.0	85.2	50.2		109.2	64.5	
3-24	24.25	27100	19940	21180	14620	349	371	256	0.0045
			73.6	78.1	54.0		106.2	73.4	
Ultimate Compressive Strength of Concrete 5830 lb per sq in									
5- 3	3.00	26200	4550	5715	4180	644	808	591	0.0076
			17.4	21.8	16.0		125.2	91.6	
5- 6	6.13	26200	8565	11330	4605	593	785	319	0.0049
			32.7	43.3	17.6		132.6	53.9	
5- 9	9.25	26200	13520	15865	11520	621	728	529	0.0067
			51.7	60.5	44.0		117.1	85.2	
5-12	12.00	26200	17615	20985	13620	624	743	482	0.0091
			67.3	80.1	52.1		119.0	77.2	
5-15	15.25	25300	22000	24420	14300	613	680	398	0.0052
			87.0	96.6	56.5		111.0	64.9	
5-18	18.13	25300	26110	26110		610	610		0.0002
			103.1	103.1			100.0		
5-21	21.00	25300	26200	26200	19450	530	530	393	0.0002
			103.5	103.5	76.9		100.0	74.1	
5-24	24.00	27100	26200	28000	21850	463	493	387	0.0099
			96.8	103.3	80.6		106.4	83.6	

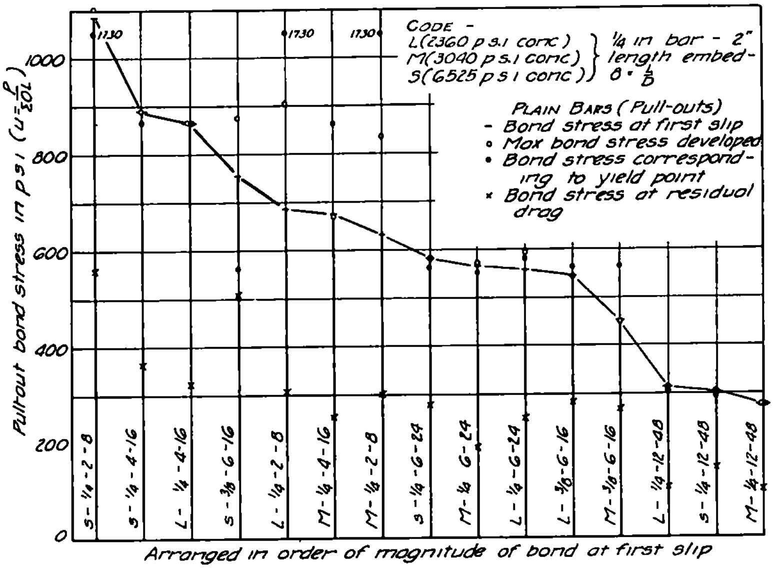

Figure 21 Length of Embedment Series of 1936 Plain Bar Pull-Out Specimens Arranged in Order of Unit Bond Stress at First Slip

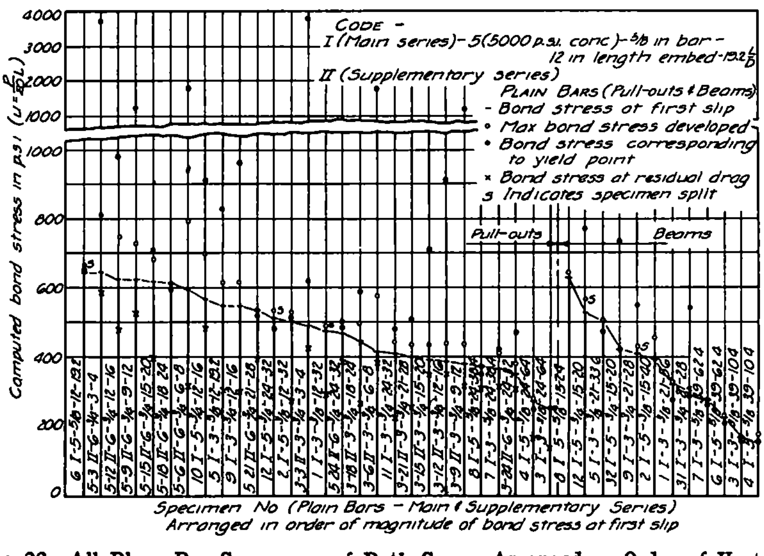

Figure 22 All Plain-Bar Specimens of Both Series Arranged in Order of Unit Bond Stress at First Slip

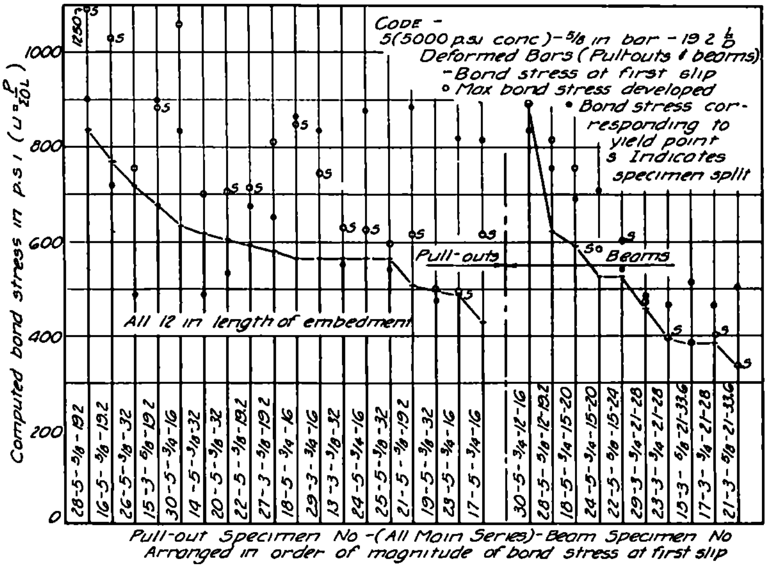

Figure 23 All Deformed-Bar Specimens Arranged in Order of Unit Bond Stress at First Slip (Note That the Supplementary Pull-Out Series Had No Deformed Bar)

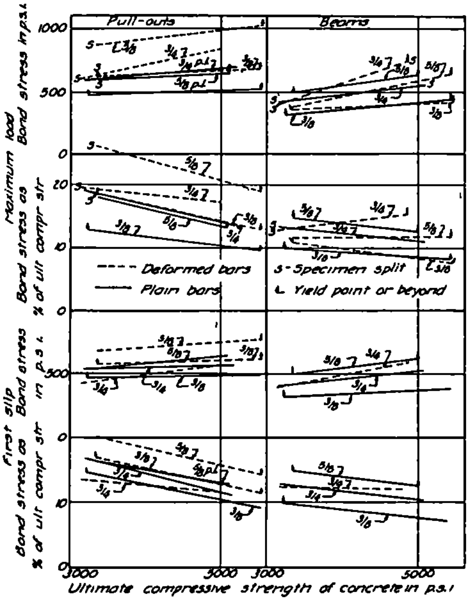

Figure 24 Bond Resistance Vs Compressive Strength of Concrete Pull-Outs with Plain Bars 2 in Cover, All Others 1½ in Cover Embed of Pull-Outs 12 in Embed of 3,000 lb Beams 21 in Embed of 5,000 lb Beams 15 in

reached yield-point stress before the ultimate load was attained and nearly all of the deformed bar specimens of both strengths of concrete split The strength of concrete seemed to have only a slight influence on the unit bond resistance developed, which is the reason for the decreasing ratio of bond resistance to strength of concrete as the strength of concrete increases

From Figure 24 the following conclusions can be drawn

1 For both the pullout tests and the beams, the bond resistance is slightly higher for the stronger concrete both at the first slip and at maximum load

2 For both the pullout tests and the beams the ratio of unit bond resistance to compressive strength decreases as the strength of concrete increases For plain bar pullout specimens the mean ratio at first slip (approximately 0 0001 in movement) ranged downward from 16 percent for 3,000 lb concrete to 12 percent for 5,000 lb concrete For the

beams the mean range was from about 13 to 10 percent[5]

3 While, as is indicated in Figure 24, several of the plain bars reached their yield point prior to initial slippage, the trends do not differ from those that slipped at stresses below the yield point This point receives more detailed consideration in connection with Figure 30

4 In nearly every case the deformed bars split the 3,000 lb concrete before reaching their yield points, but usually attained yield point stress before splitting the 5,000 lb concrete

5 Initial slippages for the deformed bars were at stresses somewhat above those for the plain bars This was true for the 1936 tests (1, p 90) and in accord with the findings of Wernisch (6), and Brown and Clark (10), but the difference was much less than that found by Teller (11, p 251-6)

On Figure 25 the 1937 supplementary (length of embedment) series and the 1937 main series are compared with one another and also with the 1936 tests on effect of strength of concrete on the bond-compressive strength ratios at first slip The ranges of concrete strength were about the same for the several series and the agreement is striking Figure 25 also shows the pronounced influence of embedment diameter ratio (L/D) on the relative bond resistance This aspect receives further consideration under "Length of Embedment"

Length of Embedment As noted above, Figure 25 shows with great consistency the effect of length of embedment upon bond, the only truly out-of-line indication being the curve of the 1937 supplementary series for the L/D of 24

Figures 26 and 27 supply readily comparable data on L/D effect for the pullout and beam specimens all of which were in the main 1937 series To the

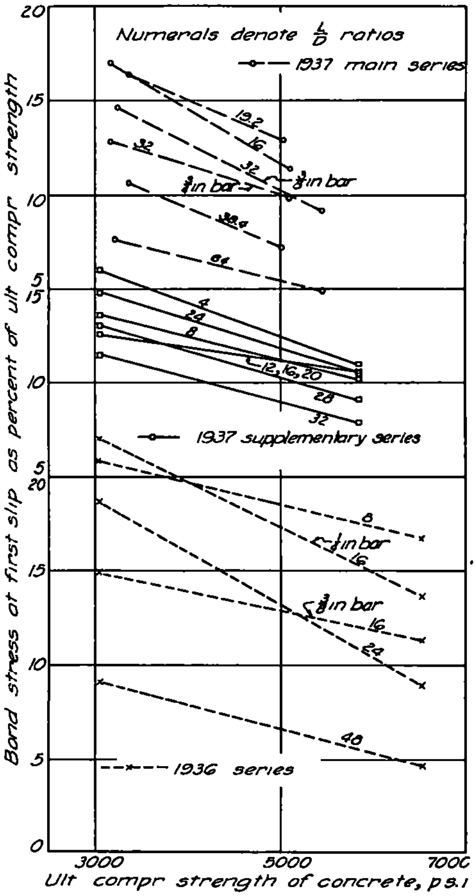

Figure 25 Bond at First Slip Vs Compressive Strength of Concrete All Bars Plain

Series	Diam of Bars	Length of Embedment
1936		
1937 Main	¼ & ⅜ in	2, 4, 6 & 12 in
1937 Supp	⅜, ⅝ & ¾ in	12 & 24 in
	¾ in	3, 6, 9, 12, 15, 18, 21 & 24 in

pullout curves of Figure 26 has been added, as a cross-tie to the 1936 tests, the same data for the 1936 mixture of the same strength The agreement is excellent, all points falling in the same

[5] Comparisons available on Figures 26 and 27 show that the differences between pullouts and beams are evidently due to their differences in L/D ratio Figure 25 shows how important the L/D ratio is in determining actual values of bond stress and of bond stress ratios

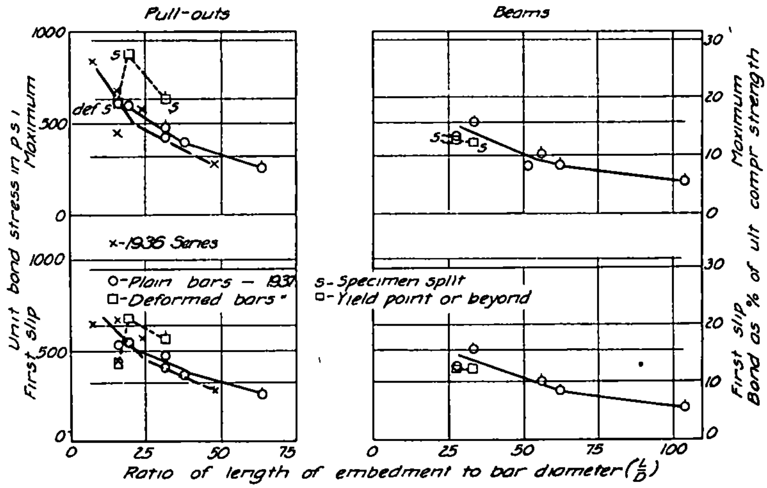

Figure 26 Main Series 3,000 lb Concrete Bond Vs Length-Diameter (L/D) Ratio at the Maximum Load and at First Slip Pull-Outs and Beam-Type Specimens Plain and Deformed Bars 1936 Pull-Outs (3,000 lb Concrete) Added for Comparison

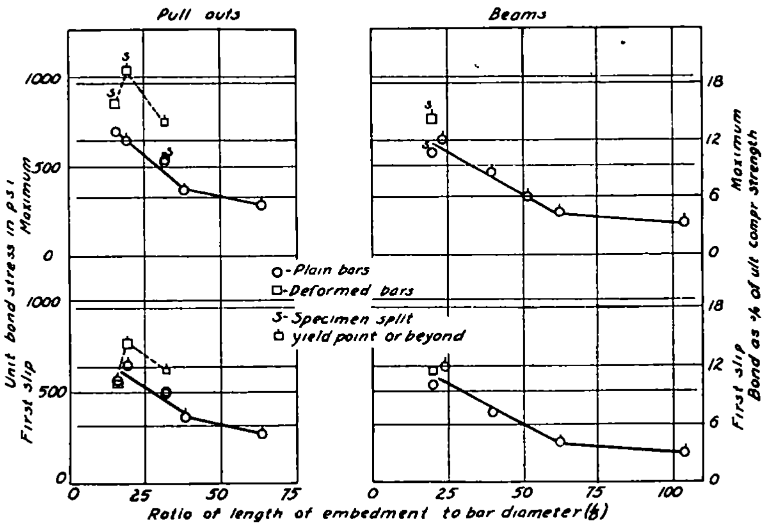

Figure 27 Main Series 5,000 lb Concrete Bond Vs Length-Diameter (L/D) Ratio at the Maximum Load and at First Slip Pull-Outs and Beam-Type Specimens Plain and Deformed Bars

zone and showing an identical trend, as would be expected from the study of Figure 25 Comparing the lower and upper groups of curves of Figures 26 and 27, there is the usual striking similarity between the behavior at first slip and at maximum load

Comparing, superficially, the pullout and the beam curves, the latter seem to parallel those for the pullouts, but are lower Actually, however, the agreement is virtually perfect in every case as can be verified by superimposing the pullout curves upon the corresponding curves for the beams The pullout curves are higher, only because they extend farther to the left, where values of L/D are lower

From the evidence of Figures 26 and 27 it is clear that, if lengths of embedment for pullout specimens and their companion beams had been the same, the plotted points on Figures 31 and 32 would have fallen almost exactly along the 45 deg line of equal beam and pullout bond strengths

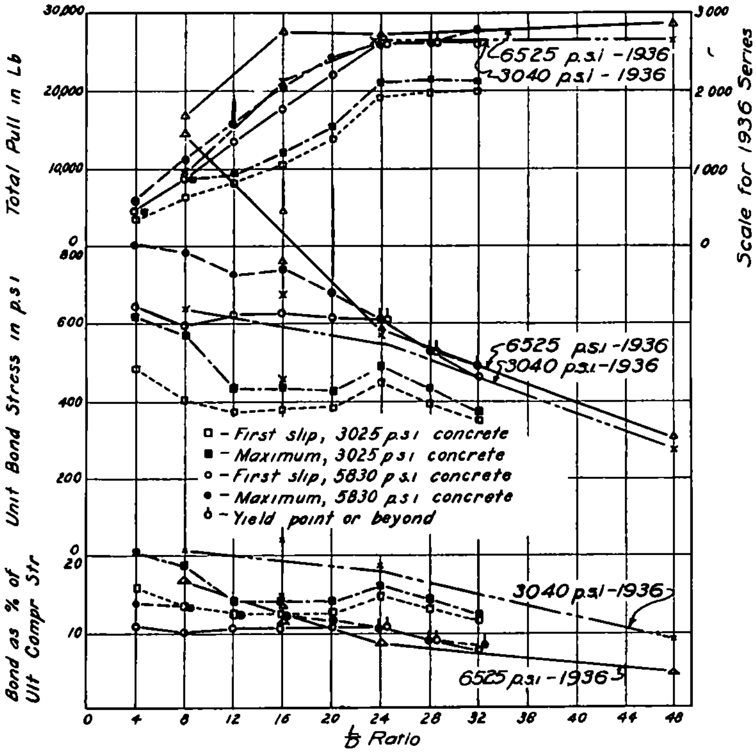

Figure 28 Effect of Length of Embedment for 1936 Series and 1937 Supplementary Series Loads, Stresses, and Bond Ratios at First Slip for 1936 Series, and at First Slip and Ultimate for 1937 Series All Bars Plain Diameters ¼ in 1936 Series, and ¾ in 1937 Series

Figure 28 supplies the data for the supplementary 1937 series in which plain ¾ in bars were embedded in pullout specimens for lengths of 3, 6, 9, 12, 15, 18, 21 and 24 in in both the 3,000 lb and 6,000 lb concretes The L/D ratio varies from 4 to 32 There were neither beams nor deformed bar specimens On the same figure are plotted the results for the two comparable-strength mixtures of 1936 for which the L/D ratio varied from 8 to 48 (1, Fig 5) These were all ¼ and ⅜ in plain bar specimens with embedments of 2, 4, 6 and 12 in For the 1936 series the loads at first slip and ultimate were practically identical (1, Table 1) while for the 1937 tests there is apparent the usual similarity between behavior at first slip and at ultimate for plain bars The following apparent facts or trends may be noted from Figure 28

1 The total resistance offered increases with added length of embedment up to about 24 bar diameters after which there appears to be little or no further increase At first thought one might attribute this to yield point stresses in the steel at this embedment, but for the 1937 tests, yield point stresses were attained only for the 6,000 lb concrete at its ultimate load, and for neither concrete was there yield point stress at first slip, nor was there such stress at ultimate for the 3,000 lb concrete That the abrupt leveling off of the total load curves at an L/D ratio of about 24 may be significant, seems probable because of the similarity to the 1936 results which were from ¼ in bars and three strengths of concrete All three strengths of concrete gave the same indication with no change in trend up to an L/D ratio of 48, the highest attained If these consistent indications are significant it means that plain bars will give added resistance as length of embedment increases up to a limiting amount[6] (24 diameters for these tests) beyond which progressive slippage will occur with attempt to apply added load, no matter how long the embedment may be This held up to 32 diameters, for these tests, 48 diameters for the 1936 tests and 64 diameters for the 1937 main series (Pullout specimens No 3 and 4, see Tables 1 and 2)

[6] A possible explanation of these observations is that at an L/D ratio of 24 or more the lower end of the bar may have slipped sufficiently, due to the accumulated elastic deformation of the steel and concrete, to have exceeded its maximum bond resistance which comes at from 0 005 to 0 01 in slip In other words a bond failure is the result of a tearing action or progressive letting go against which added length of anchorage becomes progressively less effective until some critical length (24 diameters for these tests) is reached beyond which effective resistance cannot be further distributed

2 The increase in total bond resistance with added embedment is not proportional to the amount of added embedment and the unit bond resistance reduces steadily from that for very short embedments to that for long embedments

3 The ratio of bond resistance to compressive strength reduces with increase of L/D ratio or added length of embedment as well as with increased strength of concrete as pointed out previously It should be noted that the curves for the 3,000 lb concrete have changed places with those for the 6,000 lb concrete between the middle and the bottom groups of curves on Figure 28

4 The apparent flattening out of the middle and bottom groups of 1937 curves for values of L/D between 12 and 20 is probably not significant The 1936 results show no such trend

Residual Drag: As in the 1936 tests (1, Fig 4, 6 and 8), observations were made upon the residual drag or running resistance offered by the pullout specimens of the supplementary series after considerable slippage (0 05 in in 1936 and 0 08 in in 1937 tests) Figure 29 gives these data on which the following comments are offered

1 The total residual drag for plain bars continued to increase with added embedment up to 32 diameters

2 The increase in drag was not proportional to the increased area of contact from added embedment.

3 The ratio of "drag" bond resistance to the ultimate strength of the concrete decreased with added length of embedment

4 The drag behavior was consistent and similar to the actions at first slip and at ultimate except that there was no apparent flattening out of the total load curves up to the maximum L/D ratio of 32 and there was some indication that the unit and percentage bond resistances might be approaching some constant value for large L/D ratios It seems at least apparent that a greater effective length of plain bar will function against drag than can function against initial slippage The 1936 drag test results more nearly paralleled the characteristic action at initial slip and maximum load, than do those of Figure 29

Significance or Usefulness of Drag Resistance So far as the action of reinforcement in a concrete beam is concerned, the residual drag can serve little or no useful function Reference to Columns (1) of Tables 1 and 2 will show that in practically all cases the last observed slippage prior to failure of a reinforced concrete beam was less than 0 01 in This applies to plain and deformed bars alike and it becomes obvious that a beam will have failed before the drag can aid, and the same is generally true for the lugs on the deformed bars except insofar as they raise slightly the load at first slip [7] These indications agree with those found by other authorities Figure 29 is mainly of academic interest

Influence of Stress in Steel on the Bond Resistance of Plain Bars Many of the plain bar pullout specimens of the series of 1936 reached their yield point at about the same load as that at which initial slip occurred The same was also true for some of the specimens of the 1937 series There was obviously a valid question of whether or not the slippage was the direct result of the slight drawing down of the cross-section that accompanies a yield point stress

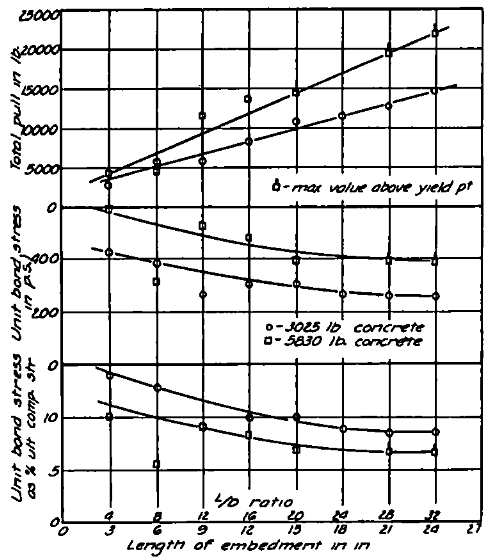

Figure 29 Residual Drag for Supplementary Series After End Slip of 0 08 in All Bars ¾ in Plain. 3,025 and 5,830 lb Concrete. Total Loads, Bond Stresses and Bond Ratios

Figure 30 seems to indicate rather definitely that even the elastic reduction in cross-section (Poisson ratio effect) has an influence upon the increment of load between first slip and ultimate, and that when the first slip occurs at a yield point stress, the load at first slip and ultimate will be identical That this latter should be true is to have been expected That each added increment of stress in the bar at the time of first slip reduces the added bond load that can be resisted, is an interesting and

[7] Probably this should be qualified for deformed bars of relatively long embedment By the time the bar has picked up slack and slipped sufficiently to show slip at the free end (that of zero tensile stress), the lugs near the end where load is applied, will probably have taken up their slack and should be functioning as lugs to aid the bond resistance that exists further along the bar The relative steepness of the left portion of the deformed bar curves of Figure 34 for the higher loads may well be explained on this basis In like manner there is probably some drag resistance of a plain bar which aids the unbroken bond resistance farther from the region of high tensile stress

unsuspected indication but one that does not seem unreasonable For all stresses below the proportional limit, the unit decrease in diameter is one-fourth the longitudinal unit strain At or above the yield point the bar deforms at constant volume and the unit diametrical decrease is about half the longitudinal unit strain Thus for plain bars stressed well below their yield points there must be enough drawing down appreciably to decrease the bond resistance which is what Figure 30 seems to indicate with fair consistency for the complete 1937 assortment of pullout tests on plain bars

Pullout Tests vs Beams One of the primary objectives of this investigation was to determine whether or not pullout

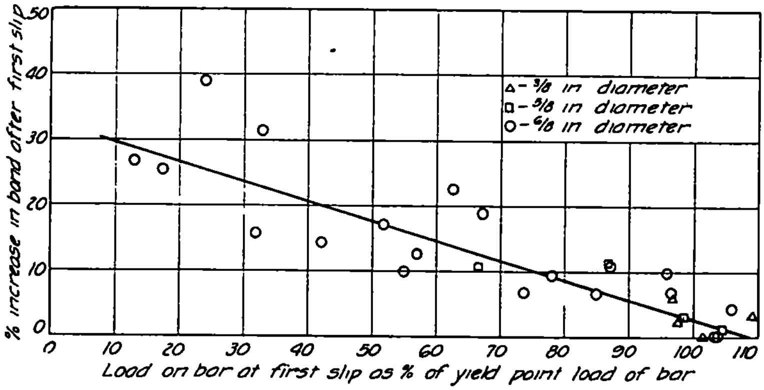

Figure 30 Effect of Stress in the Steel on the Bond Resistance at First Slip Points are Shown for All Pull-Out Specimens with Plain Bars

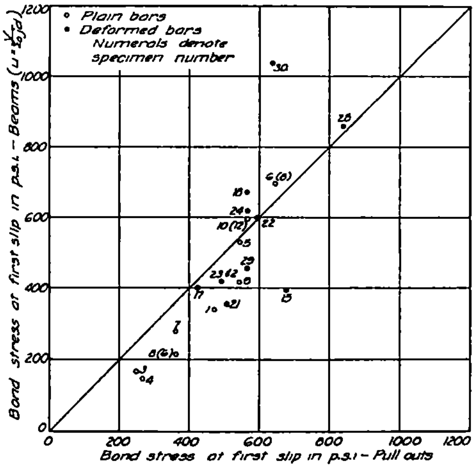

Figure 31 Beams Vs Pull-Outs at First Slip Results for All Cases That Had Companion Beam-Type and Pull-Out Specimens Several Lengths of Embedment Plain and Deformed Bars of 3/8, 5/8 and 3/4 in Diam 3,000 and 5,000 lb Concretes 1, 1½ and 2 in Cover Over Deformed Bars

tests constitute a proper criterion for the bond resistance offered by a bar in a beam

Figures 22, 23, 24, 26 and 27 all show excellent agreement between the beams and pullout tests On Figures 31 and 32 all of the companion pullout and beam tests are brought together for overall quantitative comparisons at first slip and at ultimate Very satisfying relationships are shown in spite of the numerous differences inherent in the tests

As was mentioned under the discussion of Figures 26 and 27, all of the plain bar tests would fall virtually on the 45 degree lines if the L/D values for beams were equal to those for their companion pullout specimens Values taken from Figures 26 and 27 are given in Table 4

From the almost perfect agreements shown by Table 4, it becomes apparent that for these tests the differences in L/D ratios for beams and pullouts account for virtually the entire amount of the moderate discrepancies of Figures 31 and 32, so far as the plain bar tests are concerned Since the discrepancies are no more pronounced for the deformed bars, it is reasonable to assume that the same is true for them

That in the few comparisons made by others on both pullout and beam specimens, no very satisfactory correlation casting of all specimens eliminated this variable

Some of the latest tests, those by Wernisch (6) purport to show nothing more than chance agreement between pullout and beam test results A careful study of the Wernisch data shows much better agreement than he himself evidently recognized In spite of the presence of such differences as that of

TABLE 4

Comparison of Pullout Specimens and Beams at Equal L/D Ratios
Data from Figures 26 and 27

Fig No	Strength of concrete	Stage of test	L/D	Bond stress, lb per sq in	
				Pullout	Beam
26	3000	1st slip	25	490	500
26	3000	1st slip	50	300	330
26	3000	Ultimate	25	530	500
26	3000	Ultimate	50	310	330
27	5000	1st slip	25	520	540
27	5000	1st slip	50	310	300
27	5000	Ultimate	25	550	560
27	5000	Ultimate	50	330	330

was apparent is not surprising Most of the pullout tests of others have been on short-length embedments, which, as has been so strikingly demonstrated, give much higher unit bond resistances than do long embedments such as are more likely to be present in the beams

The close agreement shown by Table 4 discounts, so far as these tests are concerned, the contention that the local compression in the concrete where the bar enters a pullout specimen creates a significant difference This is in accord with Abram's contention (1, p 94) as distinguished from that of Withey (1, p 92)

Normally beams are cast horizontally with likelihood of a poor contact between the concrete and the lower side of the bar, whereas pullout specimens are cast vertically with excellent contact all around In these tests the vertical

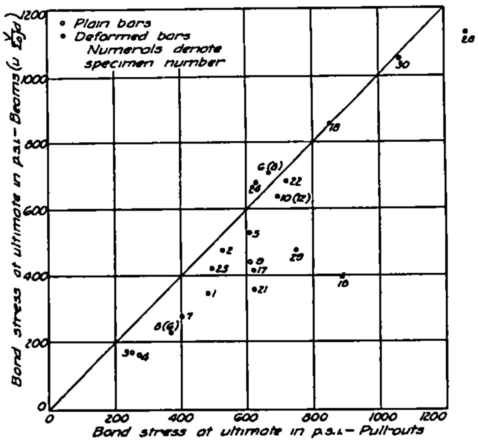

Figure 32 Beams Vs. Pull-Outs at Ultimate Results for All Cases That Had Companion Beam-Type and Pull-Out Specimens Several Lengths of Embedment. Plain and Deformed Bars of ⅜, ⅝, and ¾ in Diam 3,000 and 5,000 lb Concretes 1, 1½ and 2 in Cover Over Deformed Bars

orientation at casting, different L/D ratios and the fact that most of the tests were on deformed bar specimens, the Wernisch tests do supply excellent evidence, both at first slip and ultimate, that there is good correlation between beam and pullout indications as regards bond When all of the tests, for which there were companion beam and pullout specimens, were plotted they showed just as good, if not slightly better, agreement than do the plots of Figures 31 and 32

variably split the 4-inch square specimens, sometimes without prior first slip, and at the ultimate in nearly every instance, the only exceptions being a few $\frac{3}{8}$-in bars

Figure 33 shows that added depth of cover between 1 in and 2 in does increase the ultimate resistance of deformed bars appreciably and the first-slip resistance slightly There is nothing to indicate, however, that increased depth of cover can be justified as a device for preventing splitting from the wedging

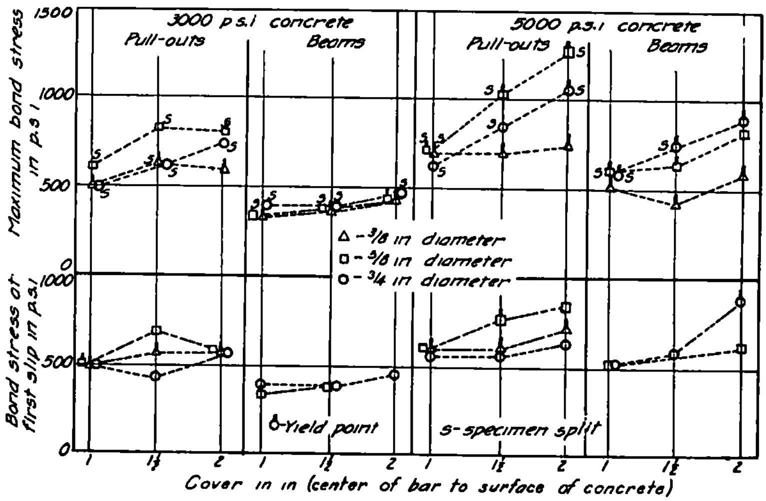

Figure 33 Bond Vs Cover for Deformed Bars. Pull-Out and Beam-Type Specimens of 3,000 and 5,000 lb Concrete Bars ⅜, ⅝ and ¾ in Covers 1, 1½ and 2 in

In the light of these tests and the Wernisch support (6), one can conclude with assurance that the pullout test can give correct indications of bond behavior for any type of bar when used in beams If at equal L/D ratios, the agreement should be actual, if not, it will be relative, as in these tests

Depth of Cover vs Splitting for Deformed Bars In the 1936 bond tests $\frac{3}{8}$-in deformed bars split the 3-in diameter pullout specimens in every instance but the $\frac{1}{4}$-in deformed bars could be pulled through without splitting For these tests the deformed bars in-

action of the lugs As might be expected the increased resistance from added depth of cover is greater for the stronger concrete The higher tensile strength should provide greater resistance against splitting, which seemed to be the case

This reconnaissance supplies added evidence of the need for the development of a deformed bar with a much finer textured and more closely spaced non-wedging roughness, and which also will come into positive bearing with less preliminary slippage than do any of the current types Wernisch (6, Fig 7) and several of the earlier investigators have demonstrated the great effectiveness of threaded bars As a practical device, threading is not in the present picture but, in such results, there is demonstrated the type of roughness that must be simulated if the deformed bar is to be made to measure up to its potentialities

Measurements of Strain As has frequently been noted the concrete in a pullout specimen, in which the bar is located centrally, is under a more or less uniform compression, the total of which equals the tension in the bar In a beam, compression, equal to the total tension in steel (plus tensile concrete), is also present The important difference is that, in the beam, all the compressive concrete is remote from the steel, whereas in the pullout specimen the tensile bar is surrounded by compressive concrete

Just how and to what extent this difference effects the interaction of steel and concrete has been speculative because convincing evidence was lacking The question is fundamental to an understanding of the bond phenomenon and the authors in planning the 1937 tests decided to extend the 1936 work of Dunagan and Ernst (7, p 96) by measuring, with Marten's mirror extensometers, on successive 4-inch gage lengths, overlapping by ¼ inch

1 Surface strains on two opposite concrete faces of all pullout specimens

2 Surface strains on the concrete of both the compressive and the tensile faces of all beam and beam-type specimens No strain observations were taken on the steel in any case

The mirror arrangements for taking strain observations have already been shown and described in connection with Figures 7-19, inclusive Complete sets of mirror readings were taken on all specimens and for all increments of loads The strain data are so voluminous that it seems desirable to present here only the limited amount deemed essential for an understanding of points covered in this paper There is in preparation a supplementary paper to be devoted almost entirely to the strain observations and their interpretation

Compressive Concrete Strains Measured at Surfaces of Pullout Specimens Figure 34 illustrates the surface compressive concrete strains on pullouts with ¾-in bars with short embedments (low L/D ratio) The 12-inch embedment for ¾-in bars gives an L/D ratio of 16 in contrast with the ratio of 64 for the 24-inch embedment of the ⅜-in bars of Figure 35 The four groups of strain curves in Figure 34 cover both strengths of concrete and plain and deformed bars

At low loads the concrete was strained only down near where the bar entered it (at the left of the diagrams) but at the high loads there was compression in the concrete throughout the entire length of the specimen There was also bond over the entire length as indicated by the slopes of the curves Wherever there is a slope the compression in the concrete is changing and that can be caused only by the presence of bond stress

Various interesting comparisons can be drawn between the curves for the plain and deformed bars as well as between those for the two strengths of concrete

Figure 35 shows plain bars with a high enough L/D ratio to produce the tearing action or progressive slippage which invalidates the assumption that bond resistance is proportional to the length of embedment When progressive slippage started at a load of about 6,000 lb, it progressed rapidly along the bars until at the ultimate there was primary bond at the top (right end) of the bar only, which was augmented but slightly by the residual bond or running friction along the bars The concrete remained compressed, because the entire load had to be transmitted through it to the bearing plate, but there is no effective bond where the curves are horizontal

There were no 24-inch pullout specimens with deformed bars so there are no companion curves for those of Fig 35 from the tensile faces of the beams are included in this paper Figures 38 and 39 are the corresponding curves from the two companion full-length beams which were tested as a check on the

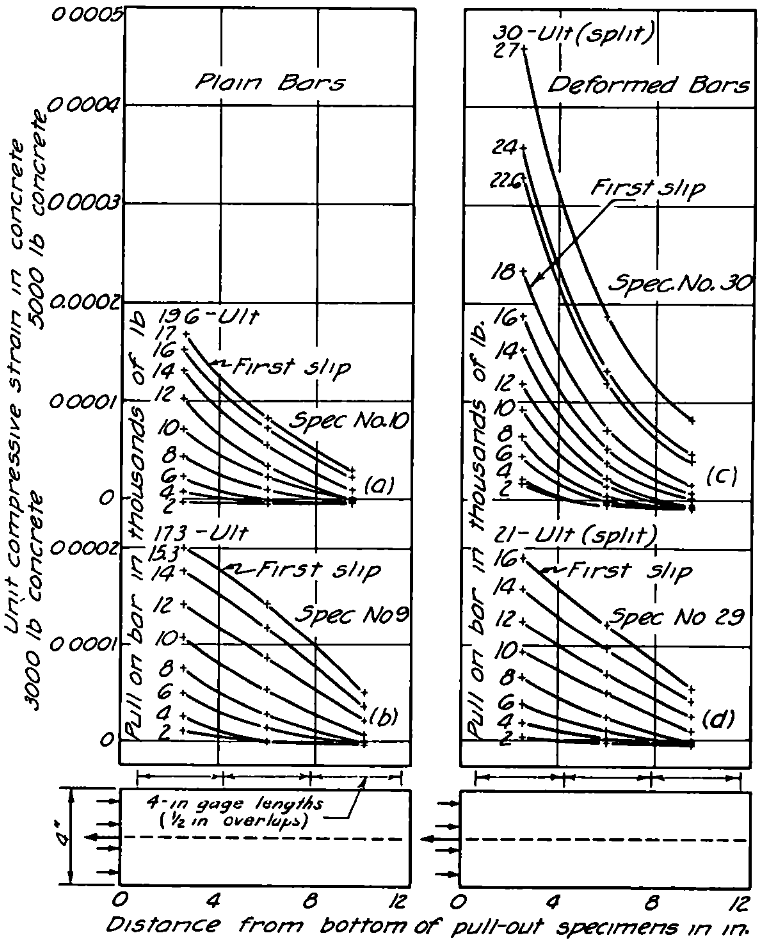

Figure 34 Distribution of Compressive Strain in Concrete for Typical 12 in Pull-Out Specimens with ¾ in Plain and Deformed Bars Plotted Points Are Shown at Centers of 4 in Gage Lengths 3,000 and 5,000 lb Concrete

Concrete Strains Measured Along the Compressive Faces of Beams Figures 36 and 37 show the plotted compressive strains for beam-type specimens No 9 and No 12 of 3,000 and 5,000 lb concrete respectively, and reinforced with ¾-in plain bars None of the strain data validity of the findings from the beam-type specimens

The qualitative agreement between the beam-type specimens and the full-length beams could not be more satisfying In both types of specimen, the region to the left of the load point (below the load point on the beam-type specimens as they were tested (see Figs 2 and 11)) is one of constant maximum moment and of zero shear, and is, therefore, a region of zero bond stress

The curves bear this out in all cases The horizontal portions show the constant values of strain for each load increment The undulations of the upper curves are due to the fact that cracks had formed on the tensile side The plotted unit strains at the proportional limit and the ultimate are from the compressive stress-strain data secured from the companion compressive control specimens

The close qualitative agreement between the right hand portions of the curves and the curves for the pullout specimens, Figures 34 and 35, are again in complete accord with expectations The compressive strain in the concrete seems to develop in the same manner as the compressive strain that is induced in the pullout specimen

Cracks and the loads at which they formed are indicated but the discussion of crack formation is reserved for treatment in connection with the detailed consideration of strain data It may be stated, however, that the mirrors along the tensile face indicated the presence of a crack at an increment or two of load prior to the time it could be detected by the eye

That each of the two full-length beams, No 31 and No 32, should have failed at a load only about two-thirds of the loads for the two companion beam-type specimens, No 9 and No 12, may have been because of the water gain effect (12) in the vertical casting of the beams, which had twice the height of the beam-type specimens, as illustrated in Figure 5 Each failed by bond and/or diagonal tension near the top end as cast That the loads were not approximately equal to those on the half-beams is deemed relatively insignificant, the important item being that the strains and apparent stress behavior were similar in all essential respects to those for the beam-type specimens, as demonstrated by Figures 36-39, inclusive

Type of Failure and Factors of Safety The authors were unable, in these tests, to distinguish between bond and diagonal tension failures, and the two are indicated together in Tables 1 and 2 When a primary bond failure occurs, the characteristic diagonal tension break follows

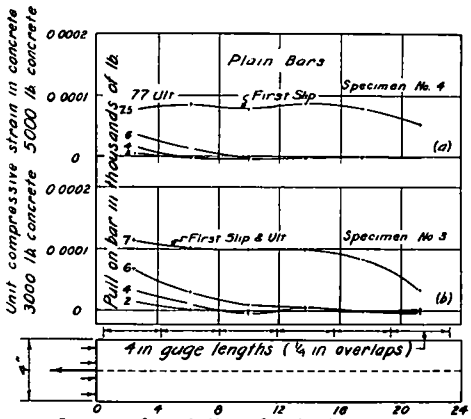

Figure 35 Distribution of Compressive Strain in Concrete for Typical 24 in Pull-Out Specimens with ⅜ in Plain Bars Plotted Points Are Shown at Centers of 4 in Gage Lengths 3,000 and 5,000 lb Concrete

at once Such failures are illustrated in Figure 15 for the beam-type specimen and Figure 18 for a full-length beam

The bottom split of Figure 15 is not significant since it is the result of the side kick that occurred from the release of compression in the rubber pad (see Figure 2) as the beam failed The entire beam below the load point is in a region of zero shear and bond stress as shown by Figures 2, 36 and 37 Such secondary bottom splitting characterized failures in bond and diagonal tension but was not present for specimens which failed by tension in the steel or for the one

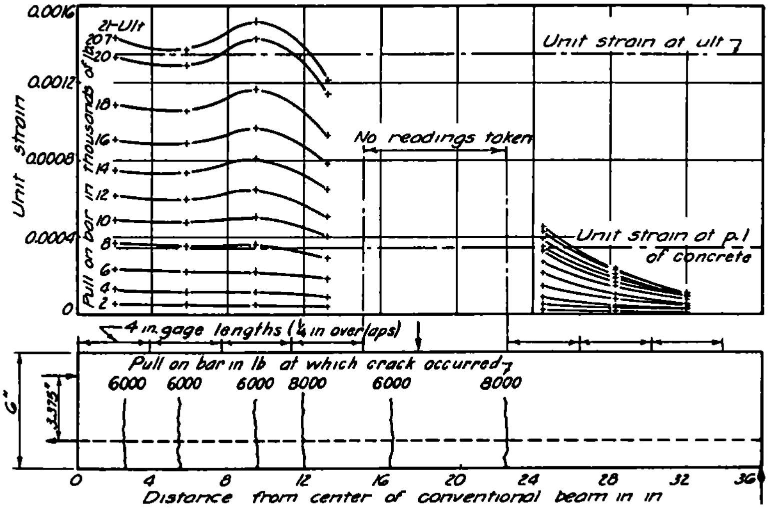

Figure 36 Strain Distribution on Compressive Face of Beam-Type Specimen No 9 3,000 lb Concrete ¾ in Plain Bar 1½ in Cover Plotted Points Are Shown at Centers of 4 in Gage Lengths

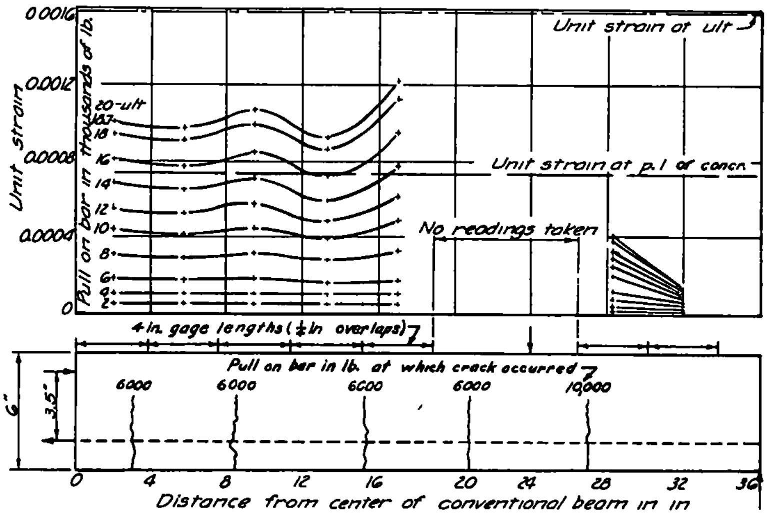

Figure 37 Strain Distribution on Compressive Face of Beam-Type Specimen No. 12. 5,000 lb Concrete ¾ in Plain Bar 1½ in Cover. Plotted Points Are Shown at Centers of 4 in Gage Lengths

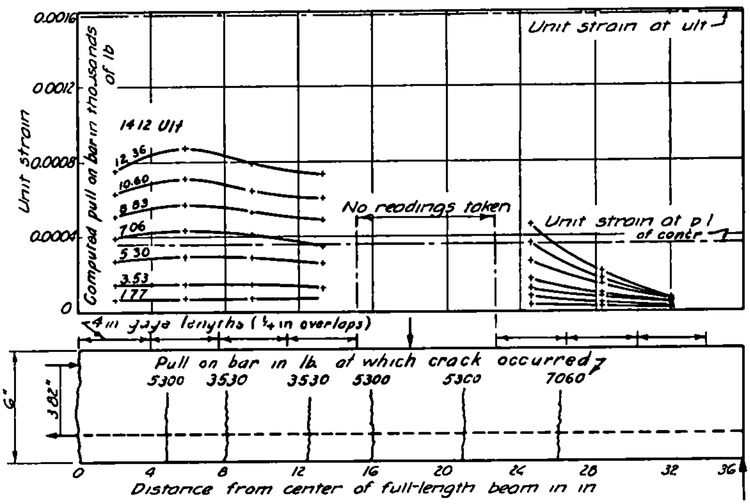

Figure 38 Strain Distribution on Compressive Face of One-Half of Full-Length Beam No. 31 3,000 lb Concrete ¾ in Plain Bar 1½ in Cover Plotted Points Are Shown at Centers of 4 in Gage Lengths

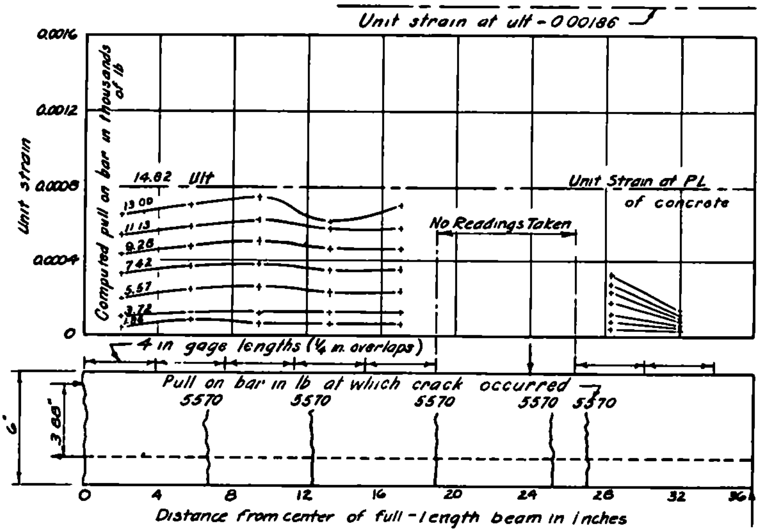

Figure 39 Strain Distribution on Compressive Face of One-Half of Full-Length Beam No 32 5,000 lb Concrete ¾ in Plain Bar 1½ in Cover Plotted Points Are Shown at Centers of 4 in Gage Lengths

specimen, No 11, which failed by compression in the concrete

Specimens No 22 and No 28 (Figure 17) failed first by tension (yield point stress) in the steel as is shown by the tensile cracks The ⅝-in deformed bars hung on enough to produce a secondary diagonal tension and/or bond failure with the characteristic top break not apparent in such other tensile failures as Beam 26 of Figure 16 In neither Beam 22 nor Beam 28 did the bottom split appear which indicates that the gradual release of energy in the rubber pad as the primary tensile failure occurred, materially diminished the violence of the "let go" at the end reinforcement In fact the factors of safety computed on the basis of current design practice (Columns w, x, y, z) are generally equal to or higher than those for failures from tension in the steel

Recalling the evidence of Figure 24, that bond increases only very slightly with increase in strength of concrete, one is surprised to note that the factors of safety for the stronger mixtures of Table 2 are higher than those for the 3,000 lb concrete of Table 1 The explanation for this lies in the cautiousness or farsightedness (whichever it may be) of those who stipulated in the current design specifications that the permissible bond stress must not go above 160 lb per sq in for plain bars and 200 lb per sq in for deformed bars, regardless of the strength of concrete This provision limits the bond stress used to that corresponding to a 4,000 lb concrete

TABLE 5

RANGES IN FACTOR OF SAFETY AT FIRST SLIP FOR ALL THE BEAMS
(See Columns w, x, y and z of Tables 1 and 2)

Concrete strength	Tension in steel	Compression in concrete	Bond	Diagonal tension
3000	2 45–3 74	4 12	2 27–3 87	2 79–4 56
5000	2 43–3 60	No failure	2 96–4 27	2 84–5 16

Full-length beams No 31 and No 32 both failed characteristically by diagonal tension and/or bond as illustrated by Figure 18 They agreed with their companion specimens No 9 and No 12 of Figures 36 and 37

It is notable that no beam with a steel percentage [8] below 1 53 for the 3,000 lb concrete and below 2 21 for the 5,000 lb concrete failed by bond and/or diagonal tension These are from 1½ to two times the percentages for balanced reinforcement at current design stresses Moreover no beam had stirrups or other web

Had the allowable loads of Column u and the factors of safety of Column y of Table 2 been computed without this restriction the factors of safety of Column y would have ranged between 2 39 and 3 49 which are not far different from the Table 1 range of 2 27 to 3 87 and decidedly below the actual Table 2 range of 2 96 to 4 27 Table 5 summarizes for ready comparison the factor of safety ranges for all of the failures of both mixtures

Table 5 indicates that for the beams of these tests the factors of safety for bond and/or diagonal tension are not out of line with those for tension It has long been known that the factor of

[8] Steel percentages are given in Col (a) of Tables 1 and 2 Note that they vary not only with size of bar but also with change in depth of cover (Col (h)) because of its influence upon the effective depth of beam Obviously the span-depth ratio of the beams is varied thereby In spite of their equal lengths, the beams were not, as reinforced concrete members, equally slender

safety for compression in reinforced concrete flexural members is high There has been, however, some question whether or not current design stresses were less conservative in bond than they should be These results seem to indicate that bond is about on a par with the others, the factors of safety in tension being lowest

Calculated Flexural Concrete Strength vs Compressive Strength As is often true the compressive stress of the concrete, as computed by current formulas (Column o), differs greatly from the observed strength secured from compressive tests, (Column b) This is in accord with many previous findings for reasons pointed out recently in this country by Whitney (8) and a few years earlier by several European investigators such as Brandtzaeg (9)

Where in Tables 1 and 2 (Column o), the computed compressive stress does not exceed by 50 per cent or more the compressive strength of the concrete (Column c), it is because the beam reached its ultimate load without stressing the concrete nearly to its ultimate For concrete beams, that fail in compression, the computed strength of the concrete will greatly exceed its true strength for the reasons referred to above

SUMMARY

1 *Beam-type Specimen vs Beam* The half-beam type of specimen performed as a beam and seemed to give results that in every way were indicative of beam behavior The major point in their favor, over the full-length beam, was the direct measurement of maximum tension in the steel instead of having to estimate by calculation the tension from known values of applied loads or from strain gage observations A minor advantage was the reduction by 50 per cent in the size and weight of the specimen to be cast, cured, and tested Some phases of the testing technique required special precautions, but placing and reading the mirrors was much simpler for the half beams

2 *Strength of Concrete vs Total Pull on Bar* For both the beams and the pullout specimens there is a slight increase in bond resistance with increased strength of concrete at first slip, and also at maximum load, but the increase is not at all proportional to the increase in strength of concrete (see Fig 24) The increase is greater for the deformed bars

3 *Strength of Concrete vs Bond Ratio* For all cases the ratio of bond resistance to compressive strength decreases as the strength of the concrete is increased (see Figs 24 and 25) Because the L/D ratio is also a major variable, percentages of decrease could be given only for a specific L/D The decrease in the value of the bond-compressive strength ratio with added strength of concrete can best be judged by a study of Figure 25 The slopes of the lines of Figure 25 are virtually all the same, the drop being 4 or 5 units between the 3,000 and 5,000 lb per sq in ordinates For example, a ratio of 0 20 for 3,000 lb concrete becomes 0 15 or 0 16 for 5,000 lb concrete, whereas a ratio of 0 12 for 3,000 lb concrete becomes 0 07 or 0 08 for 5,000 The evidence on reduction of strength ratio for added strength of concrete is in both qualitative and quantitative agreement with the reconnaissance series of 1936

4 *Length of Embedment for Pullout Specimens* For both main and supplementary 1937 series and that of 1936, the total bond resistance of plain bars increases with added length of embedment up to an L/D ratio of about 24, beyond which there is virtually no additional increase The unit bond stress decreases with added length of embed-

ment, over the full range of embedments tested, for all series Of course, the ratio of bond stress to compressive stress decreases in the same ratio as does the unit bond resistance (Figs 25, 26, 27, 28)

5 *Length of Embedment for Beams* Length of embedment-diameter ratio (L/D) effects bond resistance in beams exactly as it does in pullout specimens and in the same degree (Figs 26 and 27)

6 *Length of Embedment, Comparison with 1936 Series* The effect of length of embedment for the mixtures and small bars (¼ in) of the 1936 tests is similar in kind and amount to that for the mixtures and bars of the two 1937 series (Figs 25, 26, 28).

7 *Drag Resistance* The drag resistance of plain bars, after 0 08 in slip, was well below the bond at initial slip, and at maximum, but was similar in nature except that the total drag resistance continued to increase up to an L/D ratio of 32 (the highest for the supplementary 1937 series) The unit and percentage drag resistance continued to decrease but at a decreasing rate The drag resistance, especially of plain bars, is probably of little practical significance in beams (Fig 29)

8 *Effect of Stress in Steel on Bond Resistance* When the steel reached its yield point prior to first slip, the bond stress at first slip and at maximum were virtually identical When, at first slip, the stress was as low as 10 per cent of the yield point, the maximum bond resistance was 30 per cent higher Between these limits the increase in maximum bond stress over that at first slip was linear, as shown on Figure 30 This seems to indicate that even the elastic drawing down or "Poisson ratio" effect has its influence in facilitating progressive slippage after first slip has occurred

9 *Results from Pullouts vs Beams* Both at first slip and maximum, the pullout specimens gave results thoroughly indicative of beam test results, both for plain and deformed bars If corrected for differences in L/D ratios the agreement for these tests was virtually perfect (Figs 22, 23, 24, 26, 27, and especially 31 and 32 and Table 4) Similar tests by Wernisch (6) also show excellent agreement in spite of his failure to recognize the good correlation that his tests gave

10 *Validity of the Pullout Specimen* If correct, the foregoing conclusion validates the pullout specimen as a proper criterion of bond behavior, which, in view of its relative simplicity, is a most welcome finding In judging results from pullout or any other form of bond test, one must not lose sight of the important part played by the L/D ratio A standard pullout test should specify the embedment to be used, not as a fixed length for all sizes of bars but rather at a fixed value of the L/D ratio

11 *Local Compression Around Bar of Pullout Specimen* Obviously such an item as the compression in the concrete of a pullout specimen at point of entry is unimportant in its influence upon the validity of the test results as compared with those from a beam Even the vertical casting of pullout vs horizontal casting of beams, need not, apparently, in the light of Wernisch's tests (6), be a disturbing element

12 *Deformed Bars, Initial Slip, Spalling at Ultimate* Deformed bars gave initial slip at somewhat higher loads than did the plain bars and all bars above ⅜-in diameter invariably split the concrete, which determined the maximum resistance of the bar In 5,000 or 6,000 lb concrete, the deformed bar usually reached its yield point before splitting occurred, but rarely so for the 3,000 lb concrete, which split first Added depths of cover over deformed bars increased the resistance prior to splitting but did not prevent its occurrence (Fig 33) No

reasonable added depth of cover could be expected to make the extra resistance of deformed bars fully available in beams, even if the accompanying slippage could be absorbed satisfactorily Obviously some fine-textured non-wedging roughness is necessary if a fully utilizable type of deformation is to be devised

13 *Compressive Strains in the Concrete of Pullout Specimens* Comparisons of concrete strains, measured on the surfaces of 12-in pullout specimens, of 3,000 and 5,000 lb concrete, with plain and deformed $\frac{3}{4}$-in bars, and of similar strains for a 24-in pullout specimen, with a $\frac{3}{8}$-in plain bar, show, rather clearly, the progressive nature of the pullout action (Figs 34 and 35)

14 *Compressive Strains in Beam-Type Specimens and Beams* Plotted strains, from the compressive face of beam-type specimens of 3,000 and 5,000 lb concrete, compared with similar strains on the full-length companion beams, tested horizontally, seem, conclusively, to prove the similarity of stress and strain distribution between beam-type and full-length beam specimens (Figs 36-39, inclusive)

15 *Tensile Strains and Cracks in Beam-Type Specimens and Beams* Strains were measured along the tensile as well as the compressive faces of all beam specimens but the data are so extensive that they are reserved for subsequent presentation The study of crack data has been deferred until the tensile strains are analyzed

16 *Factors of Safety* The factors of safety for these beams, computed on the basis of current design practice, are lowest against failure by tension in the steel and highest against failure by compression in the concrete Bond and diagonal tension are well in line and differ little from one another in factors of safety (Table 5).

17 *Limitation of Maximum Bond Stresses for Design* The fact that increased strength of concrete increases bond resistance but slightly demonstrates the wisdom of limiting the maximum bond stress that can be employed in design, regardless of concrete strength

18 *Flexural Computed Compressive Strength vs Measured Strength* Where the concrete in the beams was highly stressed, the computed concrete stresses greatly exceeded the actual compressive strength of the concrete as determined by cylinder or prism tests The flexural concrete does not, of course, support the higher compression, this being but an illustration of the recognized fact that actually the stress over the cross-section of a concrete beam is not linear in distribution (8) (9)

19 *Permissible Limits of Slippage and Strains in the Steel* Tables 1 and 2 show that most beam-type specimens and beams failed at an end slippage below 0 01 in This agrees with many previous observations Amounts of slippage at the loaded end of bar was not measured in any case Such data would be valuable and should be secured in connection with future tests Direct measurement of strains on the steel would also add to the better understanding of bond action

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TESTS OF REINFORCED CONCRETE HINGES OF THE MESNAGER TYPE

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SYNOPSIS

Sixty-two reinforced concrete hinges of the Mesnager type were tested as the beginning of an investigation at the University of Maryland upon the principal hinge types used at present in rigid frame construction Bar sizes of $\frac{1}{4}$, $\frac{3}{8}$, and $\frac{1}{2}$ in were used with various combinations of rotation, thrust, and shear The bars, with l/r values of approximately 40, 65, and 90, were crossed at 15, 30, and 45 deg with the center-line of the hinge

It was found that for these hinges, (1) proper placing and wiring of the transverse steel was essential, (2) the ultimate load was practically a linear function of the number of bars, (3) rotation of the top block affected the ultimate thrust only slightly for rotations encountered in practice, (4) increase in load capacity could be obtained by the use of high yield point steels, (5) the ultimate load was reduced with increases in the l/r ratio of the bars, (6) computed stresses indicated a satisfactory agreement between tests and the equations presented by Parsons and Stang in *Proceedings*, A C I Vol 31, p 304, (7) combinations of thrust, shear, and rotation were not particularly severe unless the rotation was clockwise with shear to the right (or counter-clockwise with shear to the left)

These tests upon reinforced concrete hinges were made for the purpose of (a) supplementing present test data, and (b) corroborating the design formulas presented by D E Parsons and A H Stang in the A C I *Proceedings* Vol 31, p 304

Sixty-six hinges were cast and tested during the months of June and July in 1937 This report contains the results from 62 of the hinges of the Mesnager type with bare bars The remaining four were of the type dependent upon soil conditions for their hinging action These four were exploratory and constitute the beginning of a proposed series of tests upon special hinge types used by various designers throughout the country A final report for all hinge types is contemplated upon the completion of the proposed program

SPECIMENS AND TESTS

Specimens 1 to 6 were cast with the dimensions shown in Figure 1, but contained a rectangular spiral in each end block for transverse reinforcement These rectangular spirals proved unsatisfactory since they could not be formed truly enough to assure intimate contact with the longitudinal steel In the remaining specimens (7-62), the transverse reinforcement consisted of individual bars with end hooks These bars were wired to the longitudinal steel with 18 gage galvanized iron wire as shown in Figures 3 and 4 The effect of this change in transverse reinforcement can be noted in Fig 7, the ultimate loads showing an increase of over 30 per cent in favor of the individual transverse bars All transverse reinforcement was formed from 3/16 in round cold rolled steel

All hinges were limited to the dimensions shown in Figure 1 in order that they might be tested in the 100,000 lb Riehle testing machine available at the time Although the 20-in height did not provide the usual 40 diameters of imbedment for bond, it was felt that the $\frac{1}{4}$-in and $\frac{3}{8}$-in bars would test without slipping and that the $\frac{1}{2}$-in bars, if bearing flush with the top of the specimens, would provide satisfactory results in most cases Only one specimen (No 22) showed definite evidence of bar slip prior to failure and this was due primarily to the action of the wedge and the rotation resistance of the hinge which limited the bearing

TABLE 1

Summary of Data on Mesnager Hinges

Item	Hinge No	Longitudinal Reinforcement pl = plain def = deformed	Yield Point from tensile test (p s i)	θ (degrees)	Hinge opening (inches)	$\frac{L}{r}$ of bars	$\frac{V}{T}$	Compressive strength of concrete at time of hinge test: Top (p s i)	Compressive strength of concrete at time of hinge test: Bottom (p s i)	Total deformations of hinge opening at ultimate load: Shear (inches)	Total deformations of hinge opening at ultimate load: Compr (inches)	Mean relative rotation of end blocks (radians)	Ultimate total load (lbs)	Visible manner of failure
	a	b	c	d	e	f	g	h	i	j	k	l	m	n
1	43	4–½″ rd pl	43,600	15	4 61	39 3	0	4500	4400	0 030	0 072	0 001	30,170	Buckling of
2	44	↓	↓	↓	4 59	39 1	↓	↓	↓	44	53	18	28,980	bars
3	45	↓	↓	↓	4 59	39 1	↓	↓	↓	31	75	41	28,700	↓
4	7	↓	44,100	30	4 06	39 9	↓	3540	3400	10	37	0	26,080	Block crack
5	8	↓	↓	↓	4 25	41 7	↓	↓	↓	14	50	19	26,930	Block crack
6	9	↓	↓	↓	4 00	39 3	↓	↓	↓	59	110	31	25,500	and buck-
7	55	↓	43,500	45	3 17	39 9	↓	4840	3730	No r'd'gs at ult		2	20,530	ling of bars
8	56	↓	↓	↓	2 84	36 2	↓	↓	↓	12	111	14	22,340	↓
9	57	↓	↓	↓	3 04	38 4	↓	↓	↓	7	38	45	18,190	Block crack
10	14	↓	44,100	30	7 05	67 5	↓	3510	3440	8	39	1	17,230	↓
11	15	↓	↓	↓	7 05	67 5	↓	↓	↓	3	26	22	16,640	↓
12	13	↓	↓	↓	7 00	67 0	↓	↓	↓	28	41	47	18,690	Block crack
13	21	↓	↓	↓	9 50	89 0	↓	4300	4050	105	36	1	14,840	and buck-
14	20	↓	↓	↓	9 60	91 0	↓	↓	↓	10	21	28	14,660	ling of bars
15	19	↓	↓	↓	9 80	92 7	↓	↓	↓	43	42	36	14,310	↓
16	49	4–⅜″ rd pl	41,500	15	3 50	39 7	0	3920	3760	0 005	0 055	0 001	17,780	Buckling of
17	50	↓	↓	↓	3 54	40 2	↓	↓	↓	12	46	23	16,410	bars
18	51	↓	↓	↓	3 48	39 4	↓	↓	↓	18	59	37	15,130	↓
19	25	↓	42,900	30	2 84	37 3	↓	3480	3890	4	32	2	15,830	↓
20	26	↓	↓	↓	2 88	37 8	↓	↓	↓	17	57	10	15,590	↓
21	27	↓	↓	↓	2 85	37 5	↓	↓	↓	34	47	34	13,830	↓
22	59	↓	43,000	45	2 45	41 0	↓	4040	4520	9	65	1	12,460	↓
23	60	↓	↓	↓	2 45	41 0	↓	↓	↓	18	60	20	11,470	↓
24	61	↓	40,900	↓	2 20	37 3	↓	↓	↓	9	49	43	10,530	↓
25	39	↓	42,900	30	5 03	64 4	↓	3630	3630	0	55	0	13,220	↓
26	37	↓	↓	↓	5 21	66 5	↓	↓	↓	21	36	14	13,570	↓
27	38	↓	↓	↓	4 89	62 5	↓	↓	↓	15	50	35	11,570	↓
28	31	4–¼″ rd pl	51,000	30	3 08	58 8	0	3680	4170	0 005	0 049	0 004	8,670	↓
29	32	6 ↓	↓	↓	3 01	57 8	↓	↓	↓	47	56	1	12,750	↓
30	33	8 ↓	↓	↓	3 02	57 8	↓	↓	↓	31	45	2	17,050	↓
31	46	4–½″ rd pl	43,600	15	4 52	38 5	0 2	4500	4400	0 146	0 065	0 001	21,300	↓
32	47	↓	↓	↓	4 59	39 1	0 5	↓	↓	150	57	1	13,660	↓
33	48	↓	↓	↓	4 53	38 5	1 0	↓	↓	96	22	2	10,360	↓
34	10	↓	44,100	30	3 87	38 1	0 2	3540	3400	68	61	1	23,400	↓
35	11	↓	↓	↓	4 01	39 5	0 5	↓	↓	66	49	1	19,790	↓
36	36	↓	↓	↓	3 83	37 8	1 0	3680	4170	75	62	2	14,130	Block crack
37	34	4–½″ rd def	61,600	↓	4 04	39 7	0 2	↓	↓	92	51	2	21,130	↓
38	35	↓	↓	↓	4 05	39 8	0 5	↓	↓	72	55	2	23,870	Buckling of bars
39	12	↓	↓	↓	3 92	38 6	1 0	3540	3400	134	86	2	23,350	Block crack and buckling of bars
40	58	4–½″ rd pl	43,000	45	3 15	39 7	0 5	4840	3730	26	34	1	19,590	Buckling of bars

TABLE 1—Continued

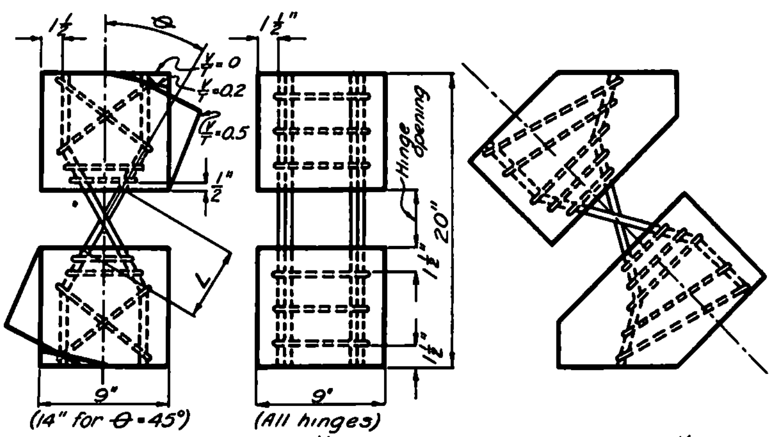
SUMMARY OF DATA ON MESNAGER HINGES

Item	Hinge No	Longitudinal Reinforcement pl = plain def = deformed	Yield Point from tensile test (p s i)	θ (degrees)	Hinge opening (inches)	L/r of bars	V/T	Compressive strength of concrete at time of hinge test: Top (p s i)	Compressive strength of concrete at time of hinge test: Bottom (p s i)	Total deformations of hinge opening at ultimate load: Shear (inches)	Total deformations of hinge opening at ultimate load: Compr (inches)	Mean relative rotation of end blocks (radians)	Ultimate total load (lbs)	Visible manner of failure
	a	b	c	d	e	f	g	h	i	j	k	l	m	n
41	52	4-⅜″ rd pl	41,500	15	3 43	39 0	0 2	3920	3760	0 131	47	0 003	10,790	Buckling of bars
42	53	↓	↓	↓	3 33	38 0	0 5	↓	↓	141	62	5	6,350	↓
43	54	↓	↓	↓	3 49	39 6	1 0	↓	↓	102	16	5	4,550	↓
44	28	↓	42,900	30	2 95	38 6	0 2	3480	3890	50	48	1	12,780	↓
45	29	↓	↓	↓	3 14	41 0	0 5	↓	↓	37	17	3	9,740	↓
46	42	↓	↓	↓	2 96	38 7	1 0	3630	3630	114	62	3	8,890	↓
47	40	4-⅜″ rd def	69,900	↓	3 02	39 6	0 2	↓	↓	27	35	1	19,570	↓
48	41	↓	↓	↓	3 04	39 7	0 5	↓	↓	93	69	1	15,970	↓
49	30	↓	↓	↓	2 89	37 9	1 0	3480	3890	126	62	2	12,880	↓
50	62	4-⅜″ rd pl	42,500	45	2 55	42 5	0 5	4040	4520	34	41	2	9,570	↓
51	16	4-½″ rd pl	44,100	30	4 01	39 5	0 2	3510	3440	52	51	29[2]	23,180	↓
52	22	↓	↓	↓	4 00	39 3	0 2	4300	4050	178	107	5[3]	18,530	Bar slip and buckling
53	23	↓	↓	↓	4 08	40 0	0 5	↓	↓	36	36	38[2]	16,660	Buckling of bars
54	17	↓	↓	↓	3 96	39 0	0 5	3510	3440	58	56	28[3]	15,740	↓
55	18	4-½″ rd def	61,600	↓	4 02	39 5	1 0	↓	↓	141	75	38[2]	20,880	↓
56	24	↓	↓	↓	4 05	39 8	1 0	4300	4050	No r'd'gs at ult		39[3]	17,550	Block crack
57	1[1]	6-½″ rd pl	43,700	30	4 20	41 2	0	2500	2500	Dials not atttached			19,650	↓
58	2	↓	↓	↓	6 00	57 7	↓	↓	↓	Dials not attached			18,000	↓
59	3	↓	↓	↓	7 75	74 0	↓	↓	↓	No r'd'gs at ult		0 001	16,850	↓
60	4	6-⅜″ rd pl	42,900	↓	3 50	45 3	↓	2970	2970	↓		1	22,150	↓
61	5	↓	↓	↓	5 38	68 5	↓	3620	3620	↓		1	17,490	↓
62	6	↓	↓	↓	6 25	79 2	↓	2970	2970	↓		3	16,000	↓

[1] The transverse steel in hinges 1 to 6 was in the form of a rectangular spiral and was wired loosely in place

[2] Counter-clockwise rotation with shear to right or clockwise rotation with shear to left

[3] Clockwise rotation with shear to right or counter-clockwise rotation with shear to left



Details of hinges with $\frac{V}{T}$ = 0, 0.2, & 0.5 *Details for $\frac{V}{T}$ = 1.0*

Note. Individual $\frac{3}{16}$ in. hooked bars (⊂⊃) for transverse steel

Figure 1 Hinge Details

area to a territory not including the ends of the bars.

The longitudinal and transverse reinforcement was constructed in units prior to placing in forms as shown in Fig. 3. The reinforcement unit was then wired in position in the forms (Figs. 3 & 4) and the lower block cast. After 18 to 24 hours had passed, the forms were inverted and the remaining end block cast. Each

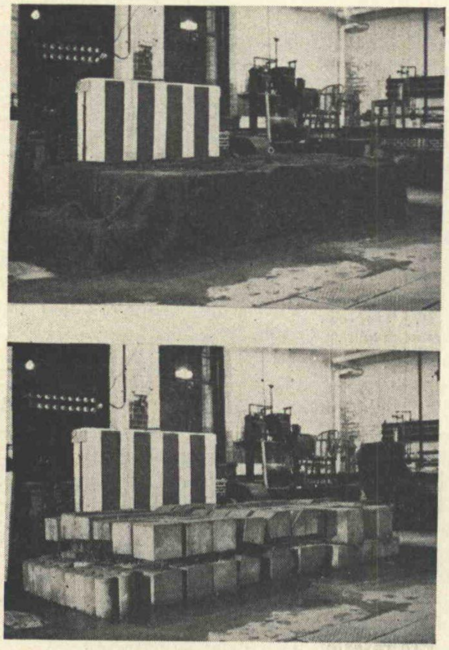


Figure 2. Method of Storing and Curing Under Wet Burlap

batch produced 6 end blocks, a full set of six hinges being cast from two batches. Two compression cylinders were made from each batch.

Six and one half gallons of water per sack of cement were used with a mix of 1:2:2 by weight. The fine aggregate was washed sand, and the coarse aggregate was a washed gravel limited to that passing the $\frac{3}{4}$-in. sieve and retained on the No. 4.

Thirty-two of the specimens were designed in groups of $\frac{1}{2}$-in. and $\frac{3}{8}$-in. bars with an 1/r of approximately 40, such that values of V/T (ratio of shear to thrust) equal to 0, 0.2, 0.5 and 1.0 were available for bar angles of 15, 30, and 45

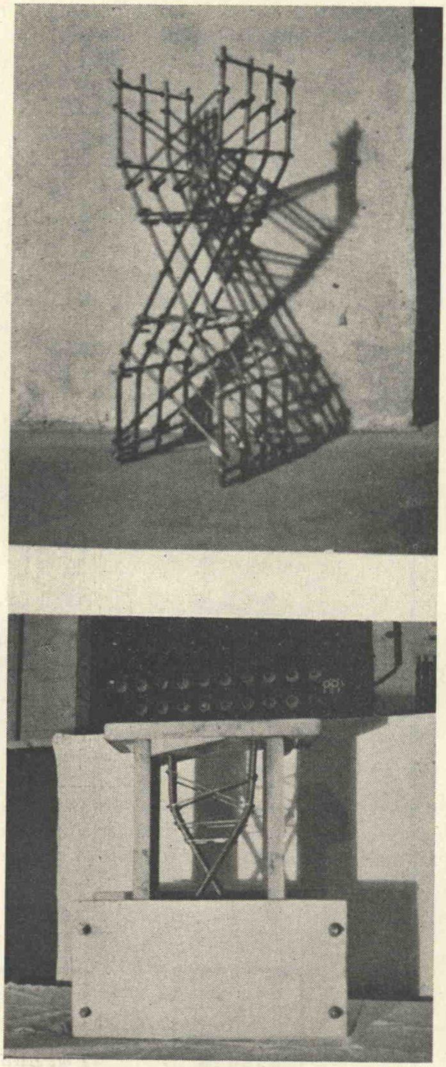


Figure 3. Reinforcement Unit for $\frac{V}{T}=0$, and Position of Similar Unit in Forms for $\frac{V}{T}=0.2$

deg. with the centerline of the hinge. The remainder, with V/T equal to zero, were cast with 1/r values of approximately 40, 65, and 90 for the purpose of obtaining the effect of rotation and width of hinge opening upon thrust. Different values of rotation were obtained by the use of 1 by 10 by 10-in. steel plates machined to wedges having slopes of 0.02 and 0.04 radians. These wedges were placed between the top block and the movable head of the testing machine. The true rotation as well as the compressive deformation was measured by means of dials bearing across the hinge opening at each corner of the hinge as shown in Figures 5 and 6. A transverse or shear dial was also used on all specimens to determine the relative shear displacement of the two end blocks.

Specimens 1 to 3 were tested after 14 days cure under moist burlap, while all other specimens were tested after 28 to 35 days under the same conditions (Fig. 2).

All specimens were tested with a thin cap of plaster of paris placed at top and bottom just prior to test. Specimens tested with wedges were capped in the machine to obtain surfaces parallel to the platen and movable head of the testing machine. A spherical bearing block was used on all specimens tested without wedges.

It should be noted that the line of action of the resultant load for $V/T=1.0$ falls outside of the area between the bars when $\theta=15°$ or $30°$ and also for $V/T=0.5$ when $\theta=15°$.

DISCUSSION OF TESTS

Transverse Reinforcement: The ultimate thrusts per pair of bars for hinges 1, 2, and 3, containing the loosely wired rectangular spiral, were 6,550, 6,000, and 5,620 lb. respectively, as compared with 13,040, 8,610, and 7,420 lb. for specimens 7, 14, and 21 in which the individual bars with end hooks were used. The concrete strengths of specimens 7, 14, and 21 were considerably higher than for the hinges 1 to 3, but the transverse component of the thrust in the bars opened cracks in both series at early loads. The severity of the cracking in hinge 1 can be noted in Figure 7 and was typical of that occurring in specimens 1 to 6 inclusive. This points to a failure of the transverse steel to function as a unit with the longitudinal steel, aided to some extent by the low concrete strength in hinges 1 to 3. It should be noted that the 1/r ratios of

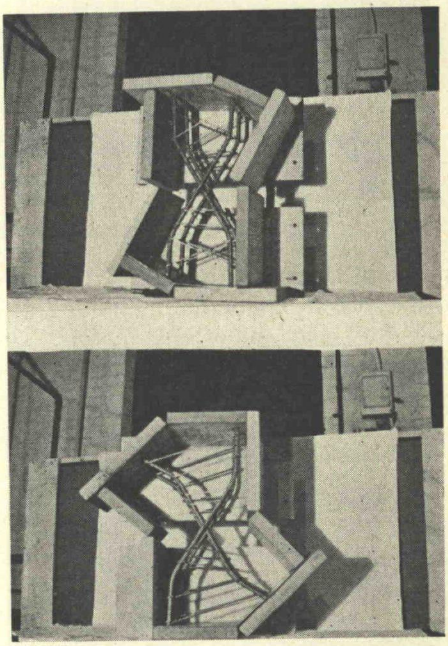


Figure 4. Reinforcement Units in Forms for $\frac{V}{T}=0.5$ and 1.0

hinges 2 and 3 were less than those of 14 and 21. This same comparison may be made between specimens 4 to 6 and 25, 39 containing $\frac{3}{8}$-in. round bars. The differences in ultimate loads are less than for the $\frac{1}{2}$-in bars, possibly because the concrete strengths and 1/r ratios were very close to the same value.

Intimate contact between transverse and longitudinal steel and the tight wiring of these points of contact, appeared to be very essential in the development of

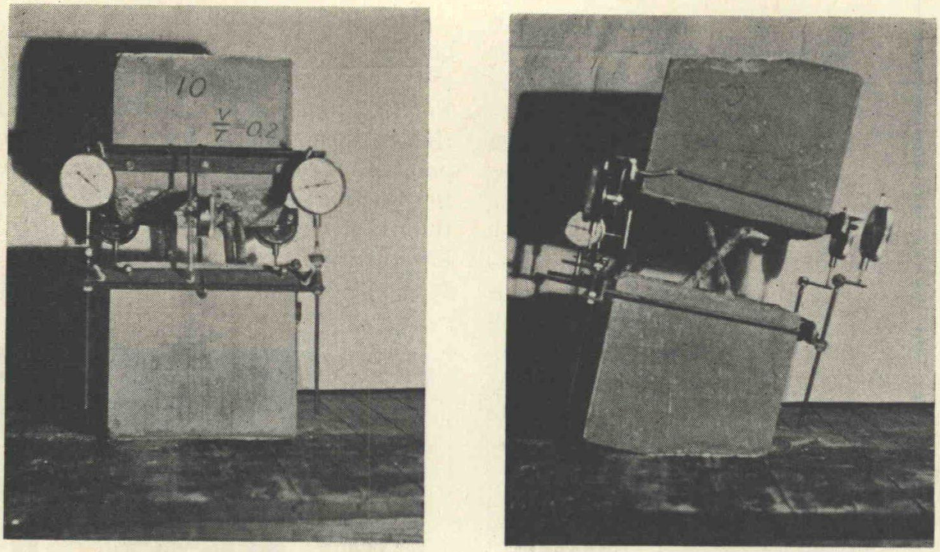


Figure 5. Front and Side Views of Dial Rigging on Test Specimen

the full carrying capacity of the bars, with concrete strength functioning as a secondary but important factor. This substantiates the recommendations made by Rear-Admiral B. Moreell, U. S. Navy (3).[1]

Number of Bars: The number of bars in hinges 31, 32, and 33 were 4, 6, and

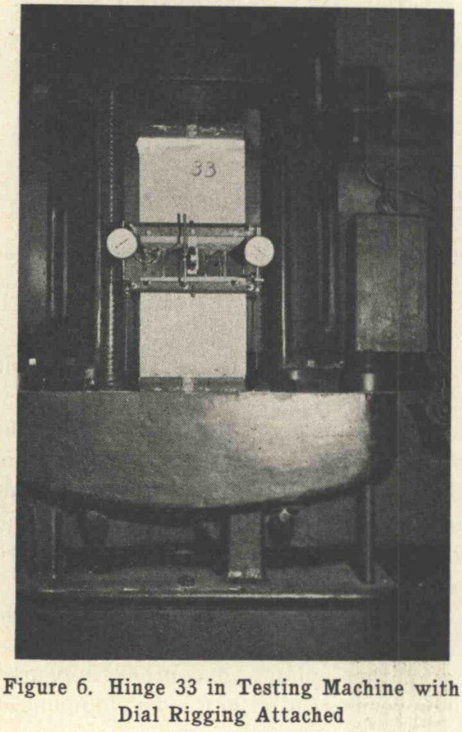


Figure 6. Hinge 33 in Testing Machine with Dial Rigging Attached

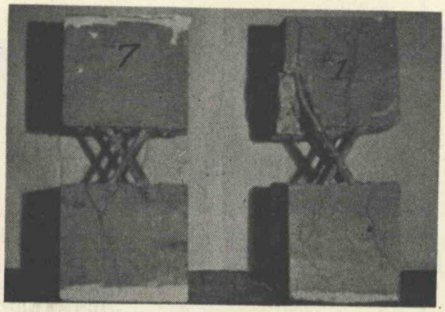


Figure 7. Improvement shown by proper placement of transverse reinforcement. Hinge 1 contained a loosely wired spiral. Hinge 7 contained individual bars wired tightly and carried to within one half inch of hinge opening. Ultimate loads were 19,650 lb. for Hinge 1, and 26,080 lb. for Hinge 7.

8-$\frac{1}{4}$ in. round, respectively. It may be noted in Table 1, as well as in Figure 12, that these three specimens showed quite satisfactory agreement in loads and ac-

[1] Figures in parentheses refer to list of references at end.

companying deformations For the size of bars used in hinges 31 to 33 it appeared that no difference would be evident between specimens with 4, 6, or 8 bars The $\frac{1}{2}$-in bar size, (hinges 7 to 9) when compared with tests by D E Parsons and A H Stang (2) give evidence that the larger number of bars provide a higher ultimate strength as well as a greater range of stiffness, although the end block cracks in hinges 7 to 9 prevent a proper comparison The four bar specimens were found to be the most convenient from a standpoint of ease of construction and rapidity of test

Rotation vs Thrust A distinct reduction in thrust is shown (Fig 8) for increasing rotations of the top block in all specimens that failed by bar buckling with the exception of group 37 to 39 A general downward trend in thrust capacity seems indicated for hinges with end block cracks although the block failures prevent an estimation of its magnitude The effect of rotations commonly encountered in practice was slight for the hinges tested

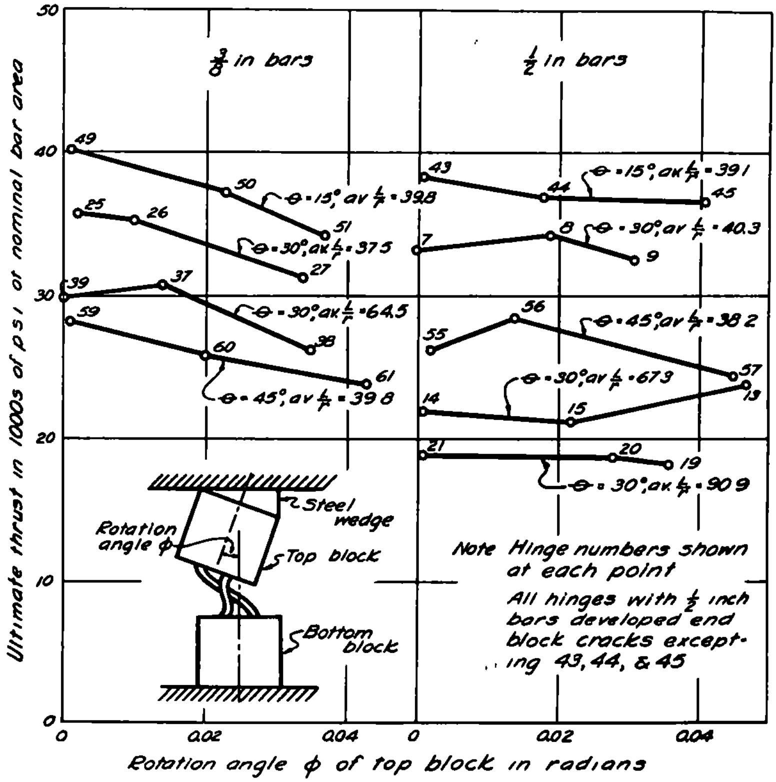


Figure 8 Rotation vs Thrust

The investigations of A Mesnager (5) and of Parsons and Stang (2) do not include hinges with bare bars tested under direct thrust without rotation Due to this, a direct comparison cannot be made between previous tests and those reported here It can be noted, however, that the reduction in thrust shown in Figure 8 for bare bars with rotation of the top block is considerably less than for covered bars with rotation of both blocks as reported by Parsons and Stang The tests of Parsons and Stang show a 21 per cent reduction in thrust for a rotation of 0 02 radians on hinges with $\frac{1}{2}$-in bars covered with mortar Figure 8 indicates approximately a 7 per cent reduction with the same increase in rotation for bare $\frac{3}{8}$-in bars

An inspection of the deformation-thrust curves in Figure 13 seems to indicate that for high rotations the hinges with $\frac{3}{8}$-in bars have an initial adjustment or rapid increase in the compressive deformation across the hinge opening at

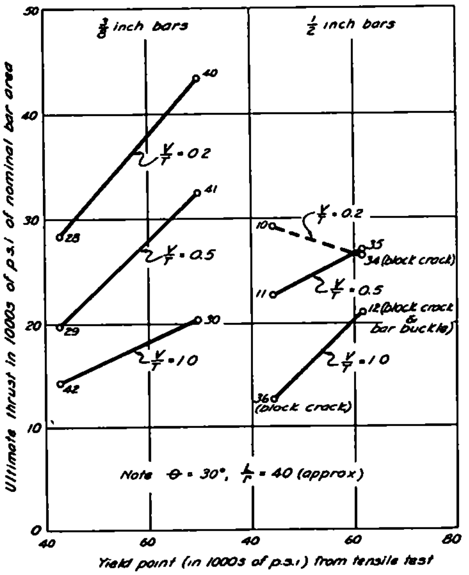


Figure 9 Yield Point vs Thrust

low loads The hinges with $\frac{1}{2}$-in bars do not exhibit this tendency

Tensile Yield Point vs Thrust The effect of increasing the yield point of the steel may be noted in Figure 9 for various ratios of shear to thrust and in Figure 15 for combinations of shear, thrust, and rotation A substantial increase, due to the higher yield point, is shown by both groups when the transverse steel is capable of taking the transverse component of the axial thrust in the bars Considering the hinges with $\frac{3}{8}$-in bars in Figure 9, the greater shear-thrust ratios show less increase due to the higher yield point This trend cannot be checked by the $\frac{1}{2}$-in bar group, due to the influence of end block cracking

In any case, it should be recognized that, in using high yield point steels, the increase in the tensile stresses in the concrete must be taken care of by properly placed and adequate transverse reinforcement

An inspection of Figure 15 for the effect of combinations of shear, thrust, and rotation, shows hinges 18 and 24 (Y P =61,600 lb per sq in) with V/T =1 0 as satisfactory as 23 and 17 (Y P =44,100 lb per sq in) with V/T=0 5 with the exception of a higher shear displacement for hinges 18 and 24 The ultimate total loads for specimens 18 and 24 exceed those for 23 and 17 (Table 1)

Further tests of hinges with bars of high yield point steel appear to be desirable in order to make certain of the trends indicated

Bar Inclination vs Thrust and Shear A bar inclination of 15 deg with the hinge center line proved to be most effective and a 45 deg angle least effective in resisting thrust without shear (Figure 10) The weakness of the 15 deg angle for shear-thrust combinations of 0 2, 0 5, and 1 0 is quite marked while the 45 deg angle is not appreciably lower than the 30 deg angle These indications are in general agreement (Fig 16 & Table 2) with the formula presented by D E Parsons and A H Stang (2) A plot of the formula shows an optimum angle between the 30 and 45 deg angles for shear-thrust ratios greater than 0 2 The optimum angles for V/T=0 5 and 1 0 might well be checked by further laboratory tests

A "bar size" effect is shown by the experimental results plotted in Figure 10 in which the $\frac{1}{2}$-in bars prove to be weaker than the $\frac{3}{8}$-in when tested under thrust alone and stronger under thrust and shear The l/r ratio of all bars was approximately 40

In general, the action of a shearing force upon the hinge is to lower its resistance to thrust, this effect becoming less for the greater ratios of shear to thrust (Figure 10)

Slenderness Ratio of Bars The effect of l/r of bars upon thrust, shown in Figure 11, is somewhat clouded by end block cracking but appears to be rather typical of column action Further tests,

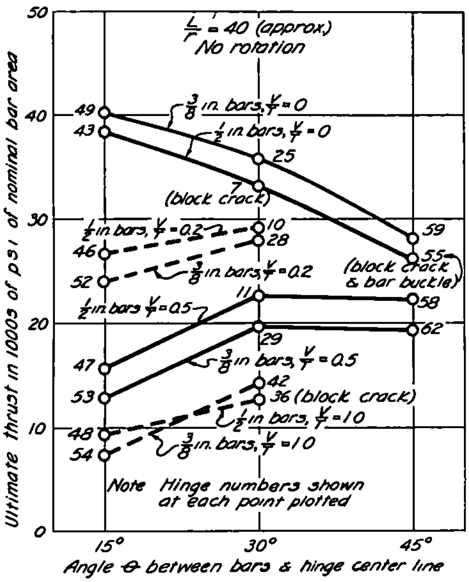


Figure 10 Effect of Bar Inclination Upon Thrust and Shear

in which end block cracks take no part, are needed in order to substantiate the trend shown

Calculated Stresses Table 2 has been prepared for the purpose of comparing stresses calculated from equations 1 and 2 (2) (presented by Parsons and Stang) with the tensile yield point stress Equation 1 assumes fixed end conditions for the bars and equation 2 assumes pin-connected ends B Moreell (3) recommends for the usual cases that the maximum stress as computed from equation 1

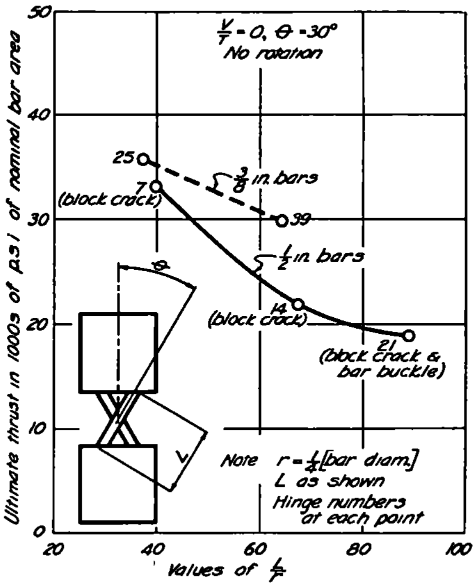


Figure 11 Effect of $\frac{L}{r}$ of Bar Upon Thrust

be kept below the tensile yield point, and that the direct stress (equation 2 without superimposed stress due to rotation) should not exceed 30 per cent of the tensile yield point

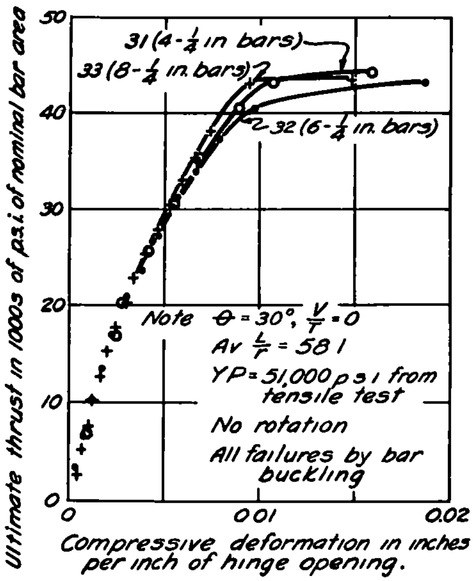


Figure 12 Deformation-Thrust Curves for Hinges with Different Numbers of Bars

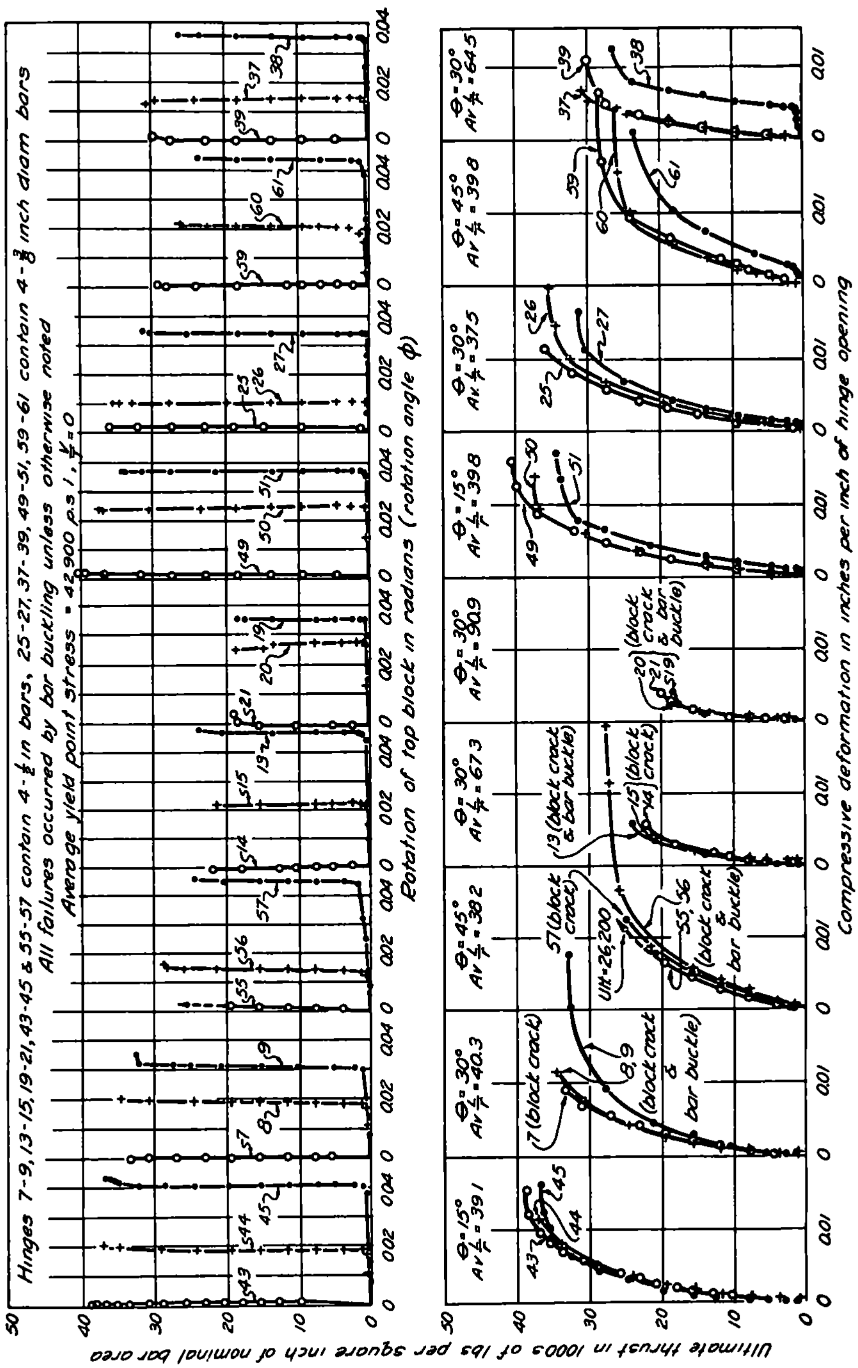


Figure 13 Deformation-Thrust Curves for Hinges Subjected to Combined Thrust and Rotation

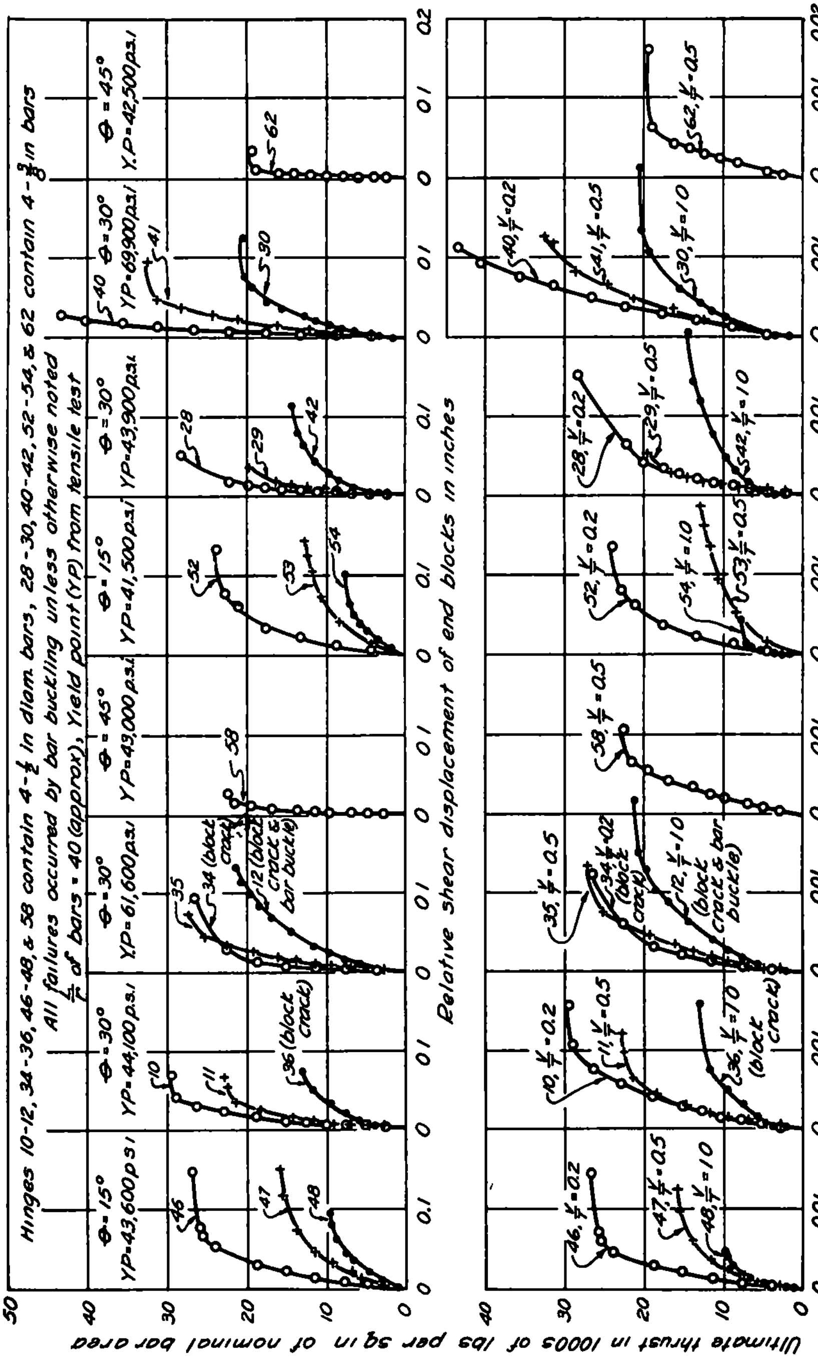


Figure 14 Deformation-Thrust Curves for Hinges Subjected to Combined Thrust and Shear

The stresses listed in the table are based on the thrust occurring at the yield load [2] Since these stresses are lower than the tensile yield point in all cases, the "set" across the hinge opening would be in excess of the 0 003 in used by Parsons without rotation than that shown by the tabulated stresses in Table 2

Figure 16 has been prepared for the purpose of comparing the theoretical curves of equation 1 with experimental curves based upon tensile yield point

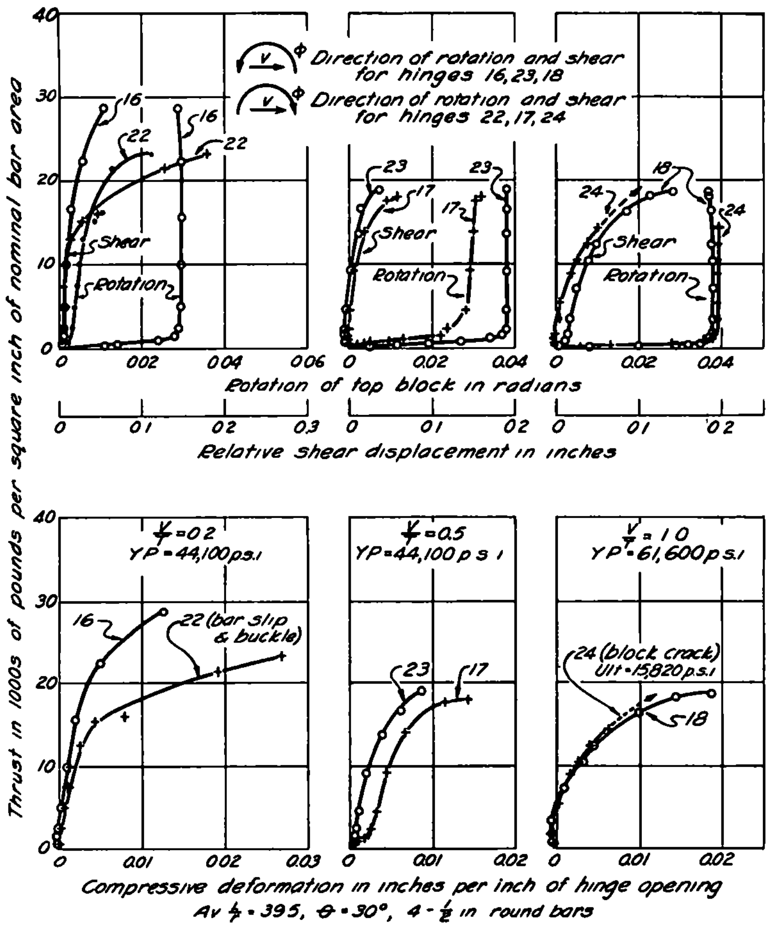


Figure 15 Deformation-Thrust Curves for Hinges Subjected to Combined Thrust, Shear, and Rotation

& Stang if the tensile yield point is used as a limit

Parsons and Stang found a closer agreement between computed and tensile yield point stress for the one hinge tested stress and thrust at ultimate and yield loads The curves resulting from the thrust at ultimate load fall below the theoretical curve, while those from the thrust at yield load fall above The curves indicate that if the stress from equation 1 be kept at or slightly below the tensile yield point, then the thrust

[2] The yield load was taken at a set in compression = 0 003 inches This value was used by D E Parsons and A H Stang See reference 2

TABLE 2

COMPARISON OF COMPUTED STRESSES WITH YIELD POINT STRESSES

Hinge	Bar size	θ	$\frac{V}{T}$	$\frac{L}{r}$	Mean rotation	Yield Load	Yield Thrust	Yield Shear	Computed stresses		Y P from tensile test
									Equation 1	Equation 2	
1	2	3	4	5	6	7	8	9	10	11	12
43	½″ rd	15	0	39 3	0 001	16750	16750	0	25200	23600	43600
7	↓	30	0	39 9	0	14000	14000	0	24100	20600	44100
55	↓	45	0	39 9	0 002	9100	9100	0	24200	19400	43500
47	↓	15	0 5	39 1	0 001	5700	5100	2550	31700	20800	43600
11	↓	30	0 5	39 5	0 001	11400	10200	5100	33200	29500	44100
58	↓	45	0 5	39 7	0 001	9600	8600	4300	26900	24800	43000
49	⅜″ rd	15	0	39 7	0 001	8100	8100	0	21800	20400	41500
25	↓	30	0	37 3	0 002	7500	7500	0	26100	22800	42900
59	↓	45	0	41 0	0 001	5500	5500	0	24300	19100	43000
53	↓	15	0 5	38 0	0 005	3300	2960	1480	39100	27800	41500
29	↓	30	0 5	41 0	0 003	6800	6090	3040	38100	34100	42900
62	↓	45	0 5	42 5	0 002	5300	4750	2370	27800	25600	42500

Note

1 Yield load was determined by "set method" of E8-33, ASTM Standards, with a set in compression of 0 003 inch across the hinge opening

2 Equations 1 and 2 are those presented by D E Parsons and A H Stang in the ACI Proc Vol 31, p 304 Equation 1 assumes fixed end conditions for the bars, Equation 2 assumes pin-connections

will be less than the ultimate but greater than that obtained by the set method at 0 003 in set for these hinges If the direct stress (equation 2 without superimposed stress due to rotation) is limited to 30 per cent of the tensile yield point, the stresses from equation 1 will fall well below the tensile yield point for the specimens listed in Table 2

Combined Thrust, Shear, and Rotation Hinges 16 to 18, and 22 to 24, were tested under a combination of thrust, shear, and rotation (Fig 15) Hinges 16 and 22, 23 and 17, 18 and 24 are comparable to hinges 10, 11, and 12 (Fig 14), respectively with a super-imposed rotation acting clockwise or counter clockwise with the shearing force The ultimate loads are lower for the specimens subjected to rotation in addition to thrust and shear, with the greater difference occurring for the higher shear-thrust ratios In all cases, a greater difference is shown for clockwise rotation acting with shear to the right (hinges 22, 17, and 24) than for counter-clockwise rotation with

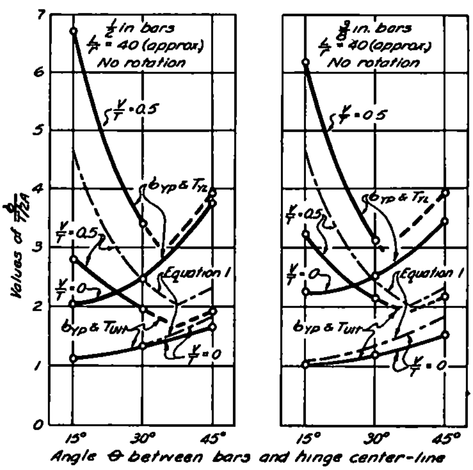


Figure 16 Comparison of Experimental Curves with Equation 1 (Reference 2).

the same shear (hinges 16, 23, and 18) That the block stresses are undoubtedly more severe for clockwise rotation with shear to the right is indicated by the fact that hinges 22 and 24 had bar slip and block crack failures respectively The mean rotation of the end block of hinge 22 (see Table 1) was no doubt affected by the slipping of the bars through the block

SUMMARY

The results from the tests of Mesnager hinges with bare bars may be summarized as follows

1 Transverse steel, even if sufficient in quantity and carried close to the hinge opening, must be wired firmly and in intimate contact with the longitudinal steel in order to develop the full thrust capacity of the bars

2 The ultimate total load was practically a linear function of the number of bars

3 The effect of rotation of the top block upon thrust carrying capacity was slight for rotations commonly encountered in practice This does not imply that stresses due to rotation are to be considered negligible (see item 6) A rapid increase or initial adjustment in the compressive deformation at low loads was apparent for the $\frac{3}{8}$-in bars when subjected to large rotations

4 A substantial increase in thrust capacity was obtained by the use of high yield point steels for all specimens in which the transverse component of the axial thrust in the bars was adequately resisted by the transverse steel

5 A marked reduction in ultimate load was shown for the higher 1/r values, although it should be noted that the results were probably affected by end block cracking

6 Computed stresses show a satisfactory agreement between tests and the equations presented by Parsons and Stang The recommendations of B Moreell (see reference 3) limiting the maximum stress to the tensile yield point appear justified and conservative

7 Combined thrust, shear, and rotation was not particularly severe unless the rotation was clockwise with shear acting to the right (or counter-clockwise with shear to the left)

ACKNOWLEDGMENTS

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Cement, sand, and gravel was furnished by the Maryland State Roads Commission Plain billet steel bars and deformed rail steel bars were furnished by the Laclede Steel Company, St Louis, Mo

Professor R B Allen of the Department of Civil Engineering of the University assisted in the testing of a large number of the specimens

REFERENCES

1 Technical Bulletin No ST28, Portland Cement Assoc June, 1937 Details and description of hinges currently used, in rigid frame design No design procedure given

2 "Tests of Mesnager Hinges" D E Parsons and A H Stang *Proceedings*, A C I, Vol 31, p 304 (1935) Tests of 7 hinges so designed as to produce failure of the hinge and to determine the effectiveness of the hinge opening to take shearing stress Design formulas were developed from a theoretical analysis of the stresses in a hinge with bare bars

3 "Articulations for Concrete Structures—The Mesnager Hinge" B Moreell, *Proceedings*, A C I, Vol 31, p 368 (1935) Description of the Mesnager Hinge and discussion of Parsons and Stang tests Charts provided for design of hinges

4 "Elastic Arch Bridges" C B McCullough and E S Thayer John Wiley and Sons, Inc, Publishers, p 330 (1931) Description of 8 tests of Considére Hinges Results show average ultimate strength of the hinges

5 "Experiences sur une semi-articulation pour voutes en Béton Armé," M Mesnager, *Annales* des Pontes et Chaussees, Vol II, p 180 (1907) Tests of two reinforced concrete hinges Each hinge 6 ft 2 in long by 1 ft 4 in deep reinforced with 13/16-in bars Elastic limit of steel was 42,600 lb per sq in The hinges were tested with an angular rotation of 0 02 radians Failure occurred outside the hinge due to insufficient transverse reinforcement, although hinge action was considered satisfactory

6 "Why Continuous Frames?" Hardy Cross, *Proceedings* A C I Vol 31, p 358 (1935) Discussion of the relative flexibility of various members in continuous structures

RELATIONS BETWEEN CURVATURE AND SPEED

BY MERRITT L FOX
General Motors Proving Ground

SYNOPSIS

In order to design the curved portions of highways properly it is necessary to study the behavior of cars on curves Both on straight and curved highways the behavior is determined largely by the side forces acting against the wheels and body

Side forces are difficult to measure on a straight highway as they are variable, being caused by road camber and side wind On a curve, the principle side force is due to the centrifugal action which can be controlled by driving the car on a circle of measured radius at constant speed This forms the basis for tests made at the General Motors Proving Ground on a large number of cars Test methods are described and typical curves given

The behavior of cars is determined by the slip angle of the front and rear wheels and the relation between them By evaluating the slip angle to the ratio of side force to weight or cornering ratio, the results of tests become applicable to curves of differing radii and to cars driven at all speeds Difference in understeering and oversteering cars is explained Understeering cars are found to handle better and require less width of lane on curves

Maximum cornering ratio is found by tests to be about 50 per cent This indicates that no highway should be constructed with a radius of curvature which will cause a car traveling at maximum speed to develop a cornering ratio above this value

Modern civilization owes much of its recent progress to more rapid transportation The worker is able to live away from the city and raise his standard of living and the farmer is dealing directly with the consumer and everyone is going somewhere The demand of progress is for increases in both the speed and safety of transportation

Automobile manufacturers have been able to combine this increase in speed with greater safety in their cars However, only in rare instances is the driver justified in using all the speed provided in his car This is due to the condition of the roads and, therefore, it would seem logical to attempt to provide highways which will permit increased driving safety at high speed To do this, a study of the behavior of cars should aid the highway engineer in his designs

Automobiles behave differently on straight or curved courses On a straight course the behavior of an automobile is principally a function of the car and the highway The car is resisted by forces which retard its progress and by other lesser forces which tend to deflect it from its line of travel The retarding forces are ascending grades, air resistance and rolling resistance, while deflecting forces are road camber and side wind

On a curved course the behavior of a car is a function of the car, the highway and the curvature The progress of the car is retarded by forces similar to those that affect it on the straight course, it is deflected by the same side forces as on the straight course plus the force due to centrifugal action As the speed on the curve is increased the centrifugal force soon exceeds the others

Forces which retard the forward progress of the car are important to the owner Any reduction in them will allow the car to be driven farther or faster for the same expenditure Side forces directly affect the handling of the car and its behavior both on curved and straight highways The side forces are therefore important to the car owner, the automobile designer and the highway engineer

The car being driven on straight or

curved highways must resist the side forces in exactly the correct amount so it will not be deflected from its course Rubber tired wheels have good resistance to side force when rolling along a straight path If we could push sidewise against the hub of such a wheel rolling along a pavement, it would require, under ideal conditions, about one hundred pounds of side force to deflect its course one degree

This characteristic of the tire is used to maintain a straight course on the highway Thus, when there is a side force of one hundred pounds acting on the wheel, it must be steered one degree toward the force to maintain its course This angle between the center line of the tread of the tire and the course maintained by the wheel is known as the slip angle

Side thrust developed by the slip angle is affected by the condition of both the tire and the roadway The tire is affected by the weight on the wheel, the tire size, the inflation pressure, the construction of the side walls, the design of the tread and the width of the wheel rim Material and finish of the road surface, the presence of foreign matter, temperature and wet or dry conditions also affect the side thrust

When side forces are encountered it is necessary for the wheels to develop sufficient side thrust to overcome the forces and hold the car on its course To do this the driver must steer toward the side from which the force is coming until the tires have enough slip angle to balance the force The rear wheels are mounted on an axle which is held more or less in alignment with the car This requires the entire body of the car to assume an angle with the course approximately equal to the slip angle of the rear wheels

On a curve the behavior of a car is largely affected by the centrifugal side force A car seldom has equal weight distribution on the front and rear ends The unequal weight will cause a larger centrifugal force at the heavier end This requires the wheels at this end to develop a larger slip angle

The centrifugal side force will cause the car body to lean to the outside of the curve and produce a transfer of weight to the wheels on the outside This will produce uneven loading on the right and left wheels both at the front and rear Since these are held in relative alignment, the side force at the front or rear end is the sum of those produced by the right and left slip angles The sum of these forces at the front or rear wheels must equal the total side thrust against that end of the car

A car in which the front end has more slip angle than the rear requires a greater steering angle than one in which the slip angles are equal This car is said to be understeering A car in which the rear has more slip angle than the front requires less steering angle and is said to be oversteering An understeering car requires the driver to steer toward the side from which the force is coming, and to increase the steering angle as the force grows larger An oversteering car requires the driver to steer away from the force

By driving a car on a circular path at constant velocity, the side force due to the centrifugal action becomes a known quantity While maintaining this velocity, measurements can be made of the steering angle, the angular position of the car on the course and the side roll By driving the car at other velocities, the side force can be controlled Thus, the measurements are made of the desired quantities at increasing values until critical readings are obtained From these measurements the quantities affecting the behavior of the car can be calculated

For the past three years, tests of this type have been made at the General Motors Proving Grounds on practically all models of American cars and the more interesting foreign cars. These tests were performed in order to study the handling charcteristics of cars and obtain data that would enable the General Motors Corporation to make them safer from the handling standpoint. No thought was given in the development of this procedure to the high speed highway. It is believed however, that the results obtained may prove useful to the highway engineer.

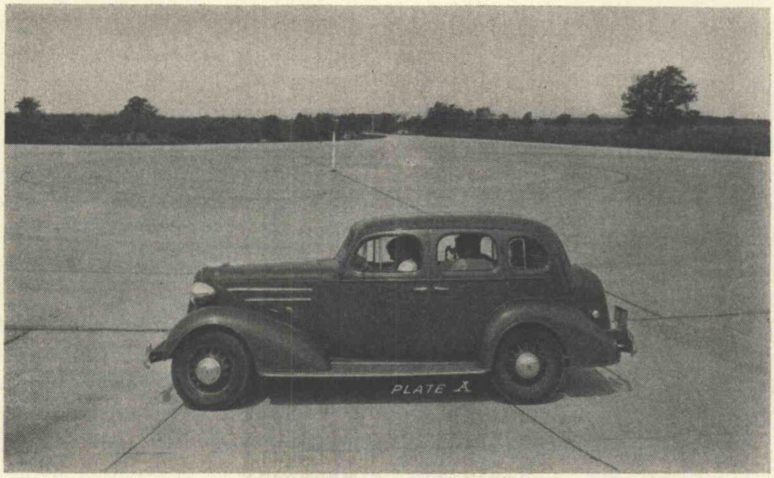


Figure 1

Figure 1 shows a car on the circle ready for the test. Preparations consist in mounting an engineer's transit and stand inside the car between the front and rear seat as near to the center line of the car as possible. An electrical device, making use of a pair of Autosyn motors connected as in signal work, is attached to the steering arm of one front wheel and wiring connections are carried to an indicator in the car. The purpose of this instrument is to indicate to the observer in the car the exact steering angle of the front wheels while the car is being driven around the circle at any given velocity.

After the tire pressure, toe-in, camber and caster are checked against manufacturers' specifications, the car is placed on the alignment equipment and a calibration is made of the steering indicator. To do this the front wheels are advanced to consecutive angular positions and the indicator read. The actual angle of each front wheel is read for each position of the steering indicator. From these readings, curves are plotted for both front wheels using actual angles turned and the corresponding indicator readings. These curves serve as a calibration of the indicator so that its readings, when driving the car in the circular path, give the actual angles of the front wheels.

Some cars are constructed with steering links of unequal length extending from steering gear on the left side of the car to the two steering arms connected to the wheels. This geometry gives unequal steering angles when the car rolls on a curve. To obtain the true steering angles in this case, it is necessary to use two sets of indicating instruments simultaneously, one connected to the right and the other to the left wheel.

In making the test runs, it is necessary to have, besides the driver, two observers in the car, one to read the steering indicator and take the speed readings, the other to set the transit angles This makes the test load equal to about four passengers The car is carefully weighed, each wheel being taken separately and the steering calibration curve made with the full test loads in place

The transit level plate is set with the car on a level portion of pavement The car is rocked from side to side to take out the error due to spring friction The transit vernier plate is set to zero by driving the car in as near a straight course as possible on the center of a straight level roadway and sighting through a telescope on a distant object at the center of the road This is repeated while driving the car at the same speed in the opposite direction The settings of the vernier plate for the two runs are averaged for the true setting Differences in settings are due to paving camber and side wind

Tests are usually made with the tire pressure recommended by the car manufacturers As the car is driven around the circle at higher speeds, the temperatures of the tires on the outside will increase faster than on the inside It is necessary to adjust the pressure as the speed is increased, since cornering power of the tires is affected by an inflation change

At the exact center of the circle to be used in the test, a small pole is erected vertically A marker band is fastened on the pole at the same height as the transit telescope

The car is now ready for the test run on the circle At the Proving Ground a circle of 108-ft radius has been used for the larger part of these tests The car is driven at a uniform speed around the circle as nearly as possible in the same track The driver must keep the speed of the car uniform and at the same time hold it on the circle The car speedometer is used only as a guide to the driver Actual speed is determined by stop watch timing of a complete lap of the circle

One observer rides beside the driver reads the steering indicator and takes the stop watch readings The second observer rides in the rear and adjusts the transit until, as seen in the telescope, the cross hairs average the travel of the band on the pole at the center of the circle By repeatedly informing the other observer and the driver as to whether the course of the car is inside or outside the exact circle as seen through the transit telescope, it is possible for the two observers to agree on the correct settings when the car is exactly on the circle

Since it is impossible to read the transit scales when the car is running, it is stopped after each run Readings of the speed and steering indicator, as well as the horizontal and vertical angle of the transit are then made and entered on the data Runs have been made at approximately $2\frac{1}{2}$ mile increments of speed, starting at five miles per hour and increasing to as high a speed as possible, consistent with holding the car on the circle and taking readings After making the test the actual speed in miles per hour is figured from the stop watch time

The actual steering angle values are taken from the calibration curve of the steering indicator

The roll angle is given by the transit vertical scale The attitude angle of the car is the angle between its longitudinal axis and a tangent to its course It is found from the horizontal angle read by the transit and the transit position angle as shown in Figure 2 The rear slip angle is the same as the attitude angle if the rear axle of the car remains truly lateral to the car as it rolls on the curve

Due to the rear spring changing position when roll occurs and one side of the car rising as the other falls, the axle moves laterally In some cars this movement of the axle is unequal at the two ends, thus causing the rear axle to steer through a small angle which either adds to or subtracts from the effective steering angle of the car The steering of the

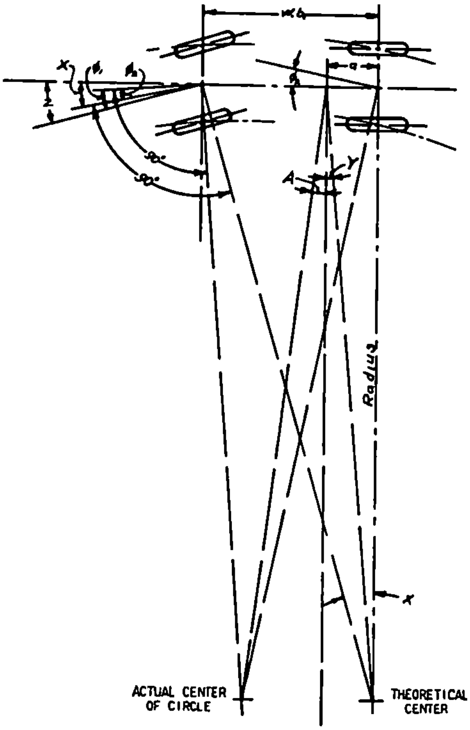


Figure 2

Z = Steering Angle Found from Tests

X = Wheelbase Angle

$$\text{Tan } X = \frac{W\ B}{\text{Radius}}$$

Y = Transit Position Angle

$$\text{Tan } Y = \frac{A}{\text{Radius}}$$

A = Angle Read by Transit

ϕ_2 = Rear Slip Angle

$$\phi_2 = Y + A$$

ϕ_1 = Front Slip Angle

$$\phi_1 = Z + \phi_2 - X$$

rear axle due to roll angle is found from another test in which the car is rolled to measured angles and the change in the position of the axle is measured In the figures shown in this report, the rear axle is assumed to remain lateral to the axis of the car

Figure 2 indicates the angular relations of a car on a circular path The theoretical center of the circle is located in line with the rear axle at a distance equal to the radius of the circle The wheelbase is denoted by w b The actual center of the circle about which the car is being driven is ahead of the theoretical center

The transit is located in the car a distance "a" ahead of the rear axle The transit vernier plate is set to read zero when the telescope points toward the inside of the circle and 90 degrees from the longitudinal axis of the car

Let A be the angle read by the transit It is negative when the line of sight is toward the rear and positive when ahead of the zero reading

Let Y be the transit position angle, the tangent of which equals the distance the transit is ahead of the rear axle divided by the radius of the circle

Then ϕ_2, the rear slip, is the sum of the angles indicated by the transit A, and the transit position angle Y

$$\phi_2 = A + Y$$

The front slip angle ϕ_1 equals the sum of the steering angle Z and the rear slip angle ϕ_2 minus the wheelbase Angle X

$$\phi_1 = Z + \phi_2 - X$$

The steering angle Z has been found from the readings of the steering indicator and the calibration curve

The wheelbase angle X is the angle at the theoretical center of the circle between lines drawn to the wheelbase (w b) of the car Its tangent is the wheelbase divided by the radius of the circle When considering the slip angle of the wheels separately, those on the outside of the circle will have the smaller wheelbase angles

The slip angles both front and rear as found by this method are the angles between the tangent to the circle and the center line of the tire at the point of contact The rear slip angle as found assumes that the rear wheels remained parallel to the longitudinal axis of the car

Figure 3 has been prepared from the readings of a test run on a standard 1937 sedan, designated as Car A The horizontal scale of speed is in miles per hour

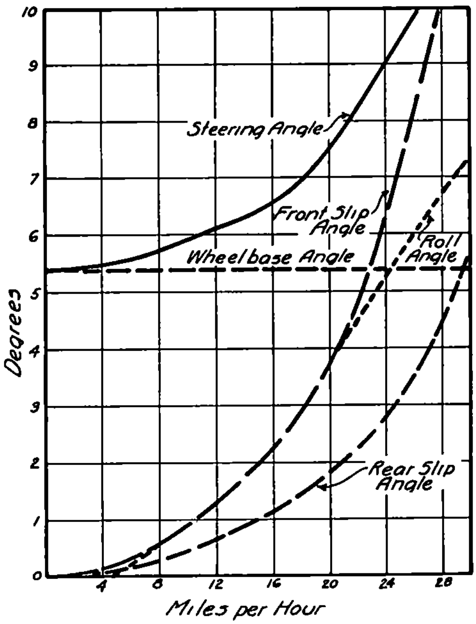


Figure 3 Angular Relations Versus Speed Car A Driven in a Circle of 108-Ft Radius

on the 108-ft radius circle The vertical scale is in degrees and applies to the different relations shown in the curves

The average steering angle of the front wheels starts near the wheelbase angle and increases with speed

Front and rear average slip angles have been extended back to zero speed The lowest test readings are made at 5 miles per hour

The roll angle crosses zero at about 4½ miles per hour This is due to the circular course having a slight side slope At this speed, the side component from the slope angle balances the centrifugal force

The curves show that the front slip angle increases faster than the rear This requires the steering angle to increase with speed in this test circle and the car is decidedly understeering

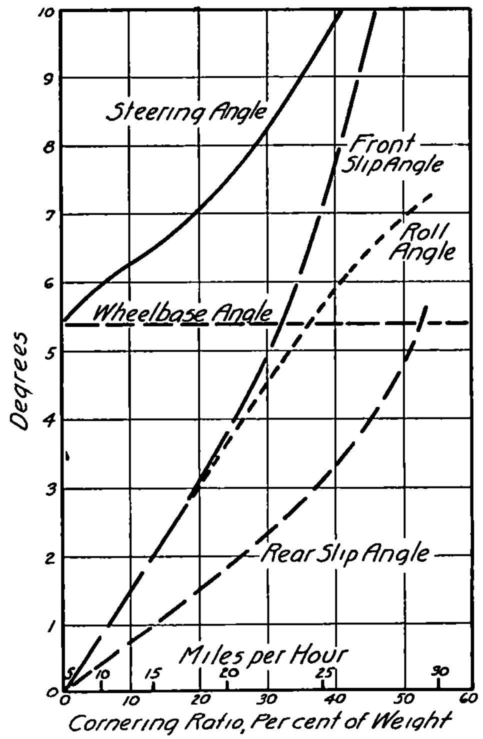


Figure 4 Angular Relations Versus Side Force Expressed as Per Cent of Weight Car A Driven in a Circle of 108-Ft Radius

Figure 4 is a replot of the quantities shown in Figure 3, but with the horizontal scale changed When the car is driven about the circle a side thrust is set up due to the centrifugal force This thrust becomes greater with increase in speed and would increase as the square of the speed except for a small slope toward the center of the circle for drainage

The formula for side force when the roadway has side elevation is

$$\text{Side Force} = \frac{W(V^2 \cos S - \sin S)}{gR}$$

In which, W equals weight in pounds
R equals radius of the circle in feet
V equals velocity of car in feet per second
g equals acceleration of gravity
S equals bank angle of the curve

In the test runs, R=108 ft, g=32 16, and S=0°21-½′
Substituting quantities

$$\text{Side Force (lbs)} = W(\ 0002875\ V^2 - \ 00625)$$

If the curves were plotted to the values of side force found by this formula, the scale would apply to cars tested on curves of equal radii These would not have as universal application as when plotted to values of cornering ratio expressed as percentage of weight

$$\text{Cornering Ratio (per cent of Weight)} = \frac{\text{Side Force}}{\text{Weight} \times 100}$$

This is the same type of ratio that Professor R A Moyer called "Coefficient of Friction" in his tests of skidding characteristics

Figure 4 uses the same vertical scale of angular values as Figure 3 with side forces expressed as cornering ratio

The scale in miles per hour on the circle of 108-ft radius has been added to retain the relationship between speed and side force from the test

Figure 5 shows the steering angles of five 1937 cars plotted in the same manner as used in Figure 4 The initial steering angle is larger for cars with longer wheelbases as will be noted from the curves

From these curves the car with the shortest wheelbase shows the greatest amount of oversteering and that with the longest wheelbase shows the largest understeering This relationship depends on the slip angles in the front and rear and not on the wheelbase Of the cars chosen for the tests, those with short wheelbases had front axles and these usually show more tendency toward oversteering

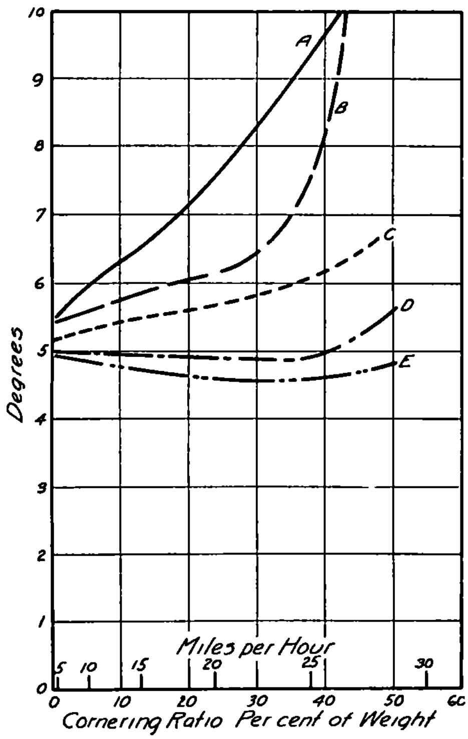


Figure 5 Comparison of Steering Angles of 5 Cars

Those with the longer wheelbases had independently sprung front wheels

Figure 6 shows the average front and rear slip angles of Cars A and E Figure 5 shows that Car A has the most understeering and Car E has the greatest oversteering This is because, in Car A, the front slip angle is larger than the rear and, in Car E, the rear slip angle is larger than the front The slip angles of Cars A and E were selected from those of the 1937 cars since these had the greatest difference between the front and rear Slip angles of other cars will plot between them

As previously stated, the values of the slip angles are affected by changes in tire characteristics and condition of the roadway as well as by the geometry of the car By plotting slip angles against values of

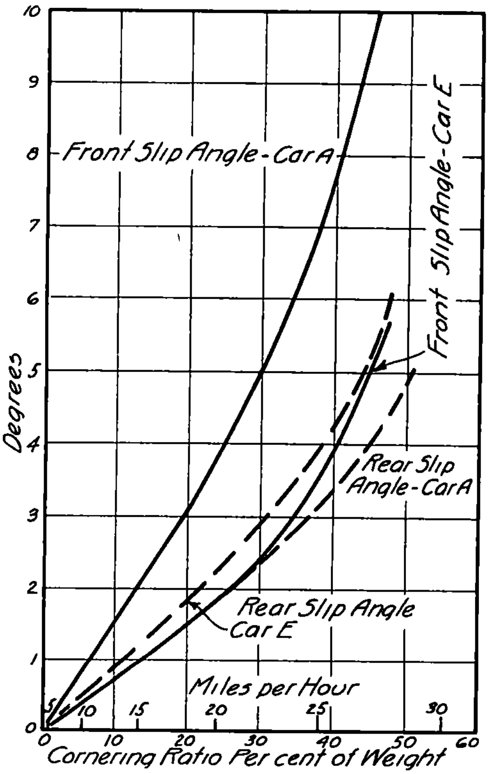


Figure 6 Comparison of Front and Rear Slip Angles on Cars A and E

side force expressed as cornering ratio, test results can be compared

Thus, a car traveling at 20 miles per hour on a flat curve of 100-ft radius has about the same centrifugal side force as one traveling at 60 miles per hour on a 1,000-ft radius

On these curves the front and rear slip angles will be much alike at the two speeds Due to the different curvature the wheelbase angle is very different, being approximately ten times as large on the curve with the small radius The steering angle will be the wheelbase angle plus the difference in the front and rear slip angles In all cases these relations may be altered somewhat In the front the steering geometry of the car may change at different radii of curvature and with greater values of rolling resistance

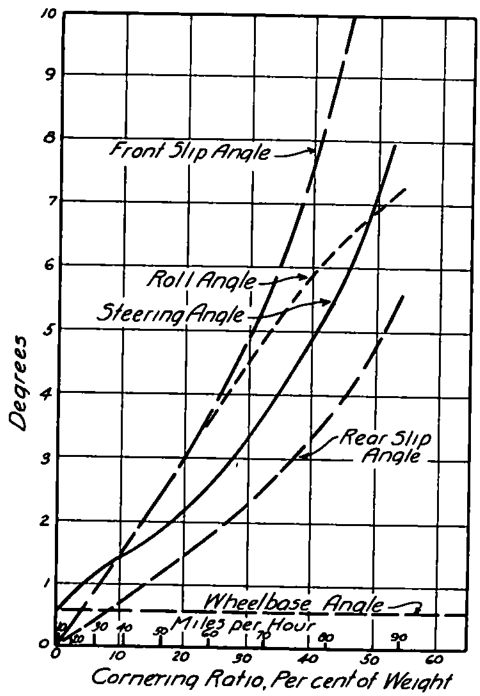


Figure 7 Angular Relations Versus Side Force Expressed as Percentage of Weight of Car A Driven in a Circle of 1,000-Ft Radius

In the rear the greater driving torque will increase the slip angle of the rear wheels by a small amount

These relations are shown in Figure 7

By the use of this method and test runs made on a curve of known radius, the handling characteristics of the car on any other curve and at any speed can be predicted

Every car will have typical curves of front and rear slip angles and of steering angle These typical curves will be repeated by the car when all the conditions of the car and the roadway are duplicated.

The circular path used in making these tests is equipped with a sprinkler system. Thus, a car can be run with the paving wet and other conditions as in previous tests.

On wet paving the slip angles are influenced to a greater degree by the condition of the tire tread than on dry pavement. A new tire will show only slightly greater slip angles up to a cornering ratio of about 40 per cent. A smooth tire will show a larger slip angle on wet pavement, the amount depending on the condition of the tread. The smoother the tread, the greater the slip angle will be.

A car being driven from a straight into a circular course has an increasing amount of side force applied at a rate depending on the abruptness of the transition. In this case, the speed is kept constant and the angle of curvature is changed.

Thus, the slip angles will increase in a manner similar to our test curves.

When the attitude angle equals $\frac{1}{2}$ of the wheelbase angle the front and rear wheels will track. At lower speed on the curve the front wheels will track outside the rear and at higher speeds they will track inside the rear. From Figures 3 and 4 these conditions can be found for Car A. The wheelbase angle is plotted at 5° 23′.

The wheelbase is then

$$1{,}296 \sin 5° 23' = 1{,}296 \times .0938 = 122 \text{ in.}$$

The front and rear wheels will track at 2° 41′, which corresponds to a cornering ratio equal to 34 per cent of its weight. This is produced by driving at 24 miles per hour on the test circle.

At approximately 30 miles per hour, the attitude equals the wheelbase angle. At this speed the rear will track $5\frac{3}{4}$ in. outside the front.

This is on Car A which is understeering. On Car E, which is somewhat oversteering, the attitude equals the wheelbase angle on the test circle when the car is driven at 26 miles per hour. At 30 miles per hour, the attitude angle will be about double the wheelbase angle and the rear will track about 11 in. outside the front.

An understeering car will require less width of highway on a curve. Including the overhang of the body beyond the wheelbase, Car E will require a lane 17 in. wider than its body. Car A requires only $8\frac{1}{2}$ in. extra width although its wheelbase is 10 in. longer.

Motor busses, freight trucks, and house trailers have long wheelbases and require a greater width of highway than passenger cars.

An understeering car such as Car A is safer than one that is oversteering. This is because the front has more slip angle than the rear and the driver has control of this end. When the front starts to skid, the driver has warning and can correct the condition. The rear end of this car will not skid before the front nor go into a spiral spin. An oversteering car may do this before the driver is aware of the dangerous condition.

From Figures 6 and 7, it is evident that the slip angles are increasing very rapidly when the cornering ratio reaches 50 per cent. It has been found impossible to drive any car above this value of side force, hold it in its course and take readings with the instruments.

At this value of side force, the tire has started side skidding, making the behavior of the car erratic. A good driver will still guide the car around the curve without trouble provided there is no traffic in the other direction.

The surface of the roadway used in the tests is somewhat smoother than the average pavement. However, with so definite a value established in many tests, it seems advisable that no curves should be constructed on main highways where a driver will exceed this value of side force when traveling at the highest speed he will attain.

The tests described are made by driving a car on a circular path. The speed is controlled to give constant centrifugal forces of known value. In this way the angular relations and the behavior of a car on a curved roadway can be studied.

A car on a straight roadway and traveling from a straight to a curved path behaves differently. In this case, the forces are variable and change direction rapidly. This sets up secondary reactions between the members of the car. The mass of the parts are carried on springs and tires which give them characteristic vibration frequencies.

The study of the behavior of cars on straight roadways is a matter requiring an entirely different test procedure, a discussion of which must be left to some future time.

CONCLUSIONS

The examples of behavior of cars on curves shown in Figures 3 to 6 were taken from tests of typical 1937 cars. In these tests the condition of the cars and their tires were carefully checked.

The amount of slip angle necessary to develop a required side thrust depends on the car maintaining its alignment, and the tires being in good condition and having the correct inflating pressures. The cars on the highways seldom meet these conditions. The front end geometry usually has been neglected and the tire pressure allowed to drop.

The behavior of cars in 1937 is better than in previous years and next year it will be still better. However, allowance must be made in highway design for cars with poorer behavior than those shown in the test curves. There are cars on our highways having more oversteering than Car E, and these require more than the 17 in. extra width of lane.

All rubber tired vehicles get their cornering power from the development of slip angles in the same way as cars. The passenger bus and freight truck have characteristic curves which are on the understeering side until the rear tires become overloaded. In this condition, the rear slip angle increases much faster than the front and the rear wheel will track a greater distance outside the front.

At least one foot should be added to the lane on curves for each 10 per cent of the maximum cornering ratio that can be developed.

In all except the most southern part of the country, sleet and ice at times may cover the highways. In this part of the country no highway should have enough superelevation so that a car will side slip into the lower lane. A reasonable coefficient of friction for ice has been suggested at 10 per cent, which corresponds to a superelevation of approximately $1\frac{1}{4}$ in. per foot.

As indicated by test results, a cornering ratio of 50 per cent is the maximum value that can be obtained under good conditions. The total cornering ratio which can be used is then the sum of that due to maximum superelevation and friction or 60 per cent. It is obviously impossible to build a highway on which it would be safe to drive a car at maximum speed when the surface is slippery. Assuming, however, that the maximum speed of 100 miles per hour is used only when the conditions are ideal, a minimum radius of 1,100 feet and a superelevation of one in ten will allow a good driver to bring the car safely around the curve.

DISCUSSION ON CURVATURE AND SPEED

Mr W C Johnson, *Goodyear Tire and Rubber Company* The work of this nature performed by Goodyear is in complete accord with Mr Fox's results This is particularly true as regards the value for maximum side skid coefficient of friction

Mr Fox has mentioned several of the factors that affect the development of side force by the pneumatic tire In view of the vital importance of the part played by the tire in this problem, I would like to amplify the description of the cornering properties of tires

Mr R D Evans, of the Goodyear Tire & Rubber Company, presented, at the January 1935 meeting of the Society of Automotive Engineers, a Paper on this subject from which I would like to quote in part

Figure 1 shows the fundamentad relation of cornering force to slip angle for a given tire and for a given set of conditions The straight portion of the curve represents that used in the normal handling of the car, and leads to the designation of the slope as the cornering power of the tire for the set of conditions indicated In this illustration the cornering power is 126 pounds per degree of slip angle

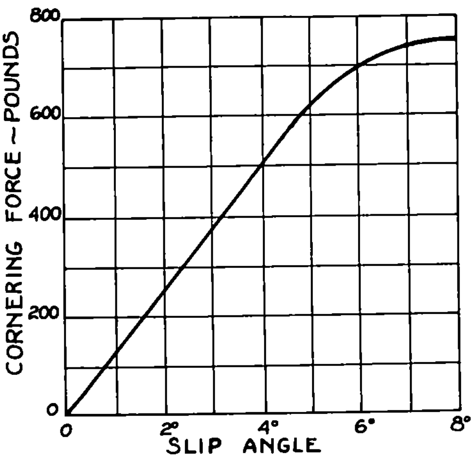


Figure 1 Tire Size 7 50-17 6 Ply Load 1500 lbs Inflation 30 lbs Camber 0°

Figure 2 gives the effect of load This is an interesting relation showing a maximum in the region of the tire's rated load The ideal and most effective tire would be one in which the ratio of cornering power to load remains a constant Then shifting loads would find the tire always able to deliver its share of cornering power This ratio has been termed the cornering coefficient

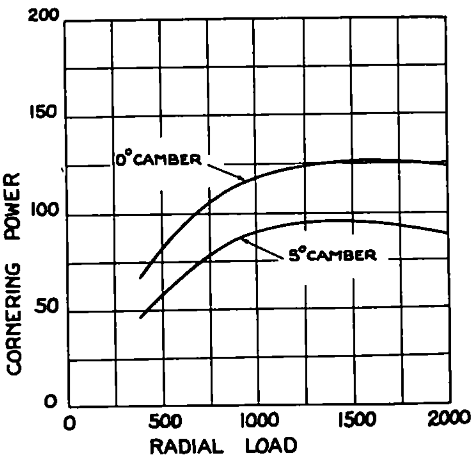


Figure 2 Tire Size 7 50-17 6 Ply Inflation 30 Lbs Per Sq In

Figure 3 shows the data of Figure 2 replotted in these terms There is now no maximum, but instead a discouraging falling off of cornering coefficient with increasing load

Figure 4 shows the extremely small effect of speed on cornering power The dotted line represents the demand on the tire by centrifugal force, which increases as the square of the speed, while the cornering power of the tire is essentially unchanged with speed This demonstrates why a tire having adequate cornering power at a low speed, may be seriously lacking in this property at a much higher speed

Figure 5 shows that increasing inflation is beneficial, tho definitely affected by the law of diminishing returns

The question of major importance is, what can be done to improve the cornering power of tires? Mr Evans in his SAE paper has referred to the "Tripod of Tire Performance" The three factors referred to are, cornering, cushioning, and durability, the tire being a compromise of all three In other words, improvement

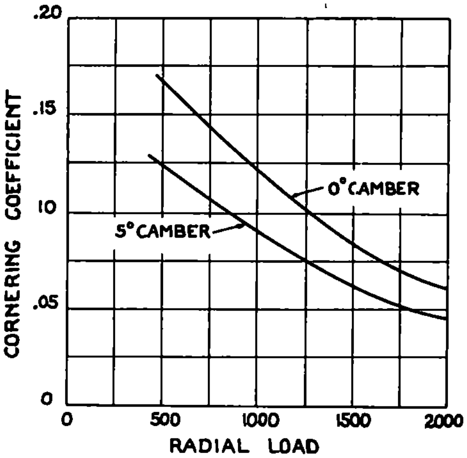


Figure 3 Tire Size 7 50-17 6-Ply Inflation 30 Lbs

of one is almost always at the expense of one or both of the others One illustration of this is the increase of cornering power gained by increasing the tire inflation This is obviously at the expense of cushioning

Larger cars require larger tires to carry the load from a durability standpoint, but larger tires are less capable of supplying cornering power in proportion to their load than smaller tires The cornering coefficients of a 5 25-17 4-ply and a 7 50-16 6-ply tire, both at their rated load and inflation, are 0 115 and 0 085 respectively

From these considerations, it appears that when heavy vehicles must be cornered at high speeds, and large tires otherwise adequate for the service lack sufficient cornering power, the substitution of dual smaller tires will result in improved handling

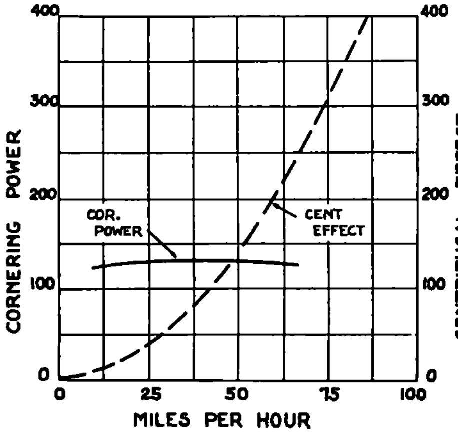


Figure 4

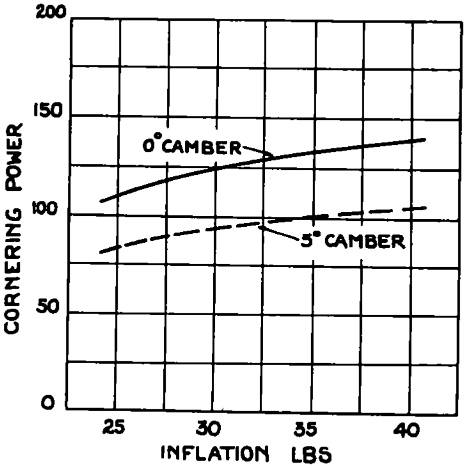


Figure 5 Tire Size 7 50-17 6 Ply Load 1500 Lbs, Rim 4 19 F

The selection of tire equipment for any automobile vehicle requires careful consideration and technical study A tire that meets the requirements of suitable durability may not be a satisfactory tire for directing and cushioning the vehicle

PROF R A MOYER, *Iowa State College* As Chairman of the Committee on "Relations Between Curvature and Speed" I should like to say how pleased I am to bring to this group Mr Fox of the General Motors Proving Ground and Mr Johnson of the Goodyear Company Too frequently designers have made assumptions that were not warranted on the basis of facts In the design of a highway, the designer should consider four distinct factors—the car, the tire, the road and the driver, and each of these factors must be studied very carefully In the work of this committee we desire to present the facts concerning the critical conditions and critical values in car operation on curves, just as the structural engineer in designing a building needs to know the critical behavior and the ultimate forces that will cause failure That is the procedure which we are now following We are trying to determine the critical behavior of the car, of the tire, of the road and of the driver, and I believe after we have examined all of those factors we will find that this matter of driving on curves is far more complicated than many of us thought it to be We also believe that after the designer has all of the facts before him, he really will be in a position to design safely for higher speeds than we are designing for now

STRESSES IN CONCRETE PAVEMENT SLABS

BY M G SPANGLER
Associate Structural Engineer, Iowa Engineering Experiment Station,
AND
FRANK E LIGHTBURN
Assistant Professor of Theoretical and Applied Mechanics, Iowa State College

SYNOPSIS

The Iowa Engineering Experiment Station has been conducting a research project for several years on the subject of stresses in concrete pavement slabs This paper is the second progress report on the project and will give the results of some extensive strain measurements from which the magnitude and direction of the principal stresses have been determined at a large number of points in the vicinity of loads applied at a corner of two full scale model slabs

The subgrade reaction pressures and deflections of one of the slabs have been measured at a number of points over the corner region and the ratio between pressure and deflection at each of the points determined These studies show that, for this slab, the ratio was constant at each point within the range of loads applied, but was not the same constant at all points since it varied with the distance of a point from the corner The values of the ratio varied from about 300 lb per sq in per in at the corner to about 75 lb per sq in per in at distances of 4½ ft from the corner The use of a uniform value of the ratio appears to be a justifiable assumption for analytical solutions, however, since average normal stresses calculated by formulas using a uniform value of 200 checked closely with average normal stresses calculated from measured strains, and also those calculated from measured pressures

The analytical determination of stresses in a concrete pavement slab when treated as an elastic plate on an elastic foundation involves a ratio between the subgrade pressure and the vertical deflection of the slab, which is usually assumed to be a constant of the same value at each point The researches of Goldbeck (1)[a] and others have shown that when loads are applied to an earth mass, the relationship between pressure and vertical movement is not a constant, but is a function of the area over which the load is applied Although a pavement slab is not a uniformly loaded area in the sense investigated by Goldbeck, it has seemed probable that the subgrade pressure may be affected by the same principle, and that the pressure at any point may be a function of position as well as deflection of the slab at that point Westergaard recognized and discussed this question in his published analysis of pavement slabs in 1925 (8)

[a] Figures in parentheses refer to list of references at end

This paper is the second progress report on some studies being conducted by the Iowa Engineering Experiment Station on the subject of stresses in concrete pavement slabs It gives the results of measurements of subgrade pressures, deflections, and principal stresses on two full scale model concrete slabs which have been studied during the past two years In both cases the measurements have been confined to the region adjacent to a load concentrated over a small area and applied at a corner of the slab The slabs have been numbered 3 and 4 for reference

The subgrade pressures were measured by means of Goldbeck pressure cells cast in the concrete In slab No 3, an attempt to measure subgrade pressure was wholly unsuccessful due, apparently, to the fact that no provision was made for initial adjustment of the pressure between the cells and the subgrade

For slab No 4, a device was designed and installed which permitted adjustment of the initial subgrade pressure

With this adjustment the cells yielded results which were remarkably consistent for measurements of this type

Deflections of the slabs were measured by means of 0 0001-in Federal dials mounted on an I-beam framework supported independently of the slabs

Strains in the top surface of the slab were measured by means of pairs of optical lever extensometers of the type described by Spangler (6), except that the fixed knife edge of each extensometer was made adjustable, a modification which greatly facilitated the initial adjustment of the mirrors The surface strains were measured along three gage lines at each of a large number of points distributed over the corner region The principal strains and stresses at each point were calculated from these tri-axial strain readings

All of the experimental slabs were located in the basement of the Experiment Station Laboratory where there was little variation in temperature They were constructed on a yellow clay subgrade which was confined in a box 14 ft square and 2 ft deep The clay was placed and hand tamped in layers of about 5 in No attempt was made to determine the properties or moisture content of the clay, although provision was made to hold the moisture content as nearly constant as possible during the test period Slab No 3 was constructed on a subgrade which had been in place for three years, and it had been allowed to dry out until it was unduly stiff Therefore it was decided that the clay should be taken out, moistened and retamped for the subgrade of slab No 4

TABLE 1

	Slab No 3	Slab No 4
Constructed	June 18, 1935	June 24, 1936
Tested	7/5/35 to 9/20/35	7/15/36 to 8/15/36 1/9/37 to 2/23/37
Size	10 x 12 ft	12 x 12 ft
Thickness	6 in	6 in
Reinforcing	None	None
Cement	High early strength	High early strength
Coarse aggregate	Limsetone	Limestone
Mix by weight	1 4 4	1 4 4
Water-cement ratio by weight	0 80	0 75
Control specimens	Cylinders and beams	Cylinders and beams
Curing	Under moist burlap	Under moist burlap
Average Properties at Time of Test		
Compressive strength, lb per sq in	3,300	4,700
Modulus of rupture, lb per sq in	520	680
Modulus of elasticity, lb per sq in	2,770,000	4,000,000
Poisson's ratio	0 20	0 25

Table 1 gives some of the more pertinent information on the third and fourth slabs

To avoid confusion and misinterpretation, a number of terms used in this paper are defined as follows

Principal stresses are the stresses at a given point which occur at right angles to planes on which the shearing stress is zero One of these is the maximum and the other the minimum stress at the point

Principal strains are the strains in the direction of the principal stresses and are the maximum and minimum strains at a given point

Rosette strains are the strains measured on three different gage lines passing through a given point

Radial strain is the strain along a line which passes through the corner

Normal stress is the stress at a given point in the slab which is at right angles to a line perpendicular to the corner bisector, or in other words a stress which is parallel to the corner bisector This is the stress which Westergaard (8) assumed to be uniformly distributed over a line perpendicular to the corner bisector

Average normal stress is the stress which is the average ordinate of a curve showing the actual distribution of the normal stress over a line perpendicular to the corner bisector

Apparent stress is a factitious stress which is obtained by multiplying the unit radial strain by the modulus of elasticity

In this paper the terms stress and strain are used to denote unit stress and unit strain unless otherwise noted

STRESS MEASUREMENTS ON SLAB 4 [b]

In order to determine completely the stresses at a point in the surface of a slab it is necessary to measure strains on three different gage lines through the given point The points on slab No 4 at which rosette strain readings were taken are shown in Figure 1 These points will be designated by giving the number and letter of the lines intersecting at the point as A-1, B-7, etc The gages had a length of 3 in and they were centered over the points with the exception of the points at the edge, where they were placed as nearly over the points as the proximity to the edge would permit

Load was applied in increments, and strain readings taken at 3,000, 4,000, and 5,000 lb Within the loading range the strain readings at a given point were directly proportional to the load The optical lever extensometers used to measure these strains were arranged in pairs on each gage length in order that the

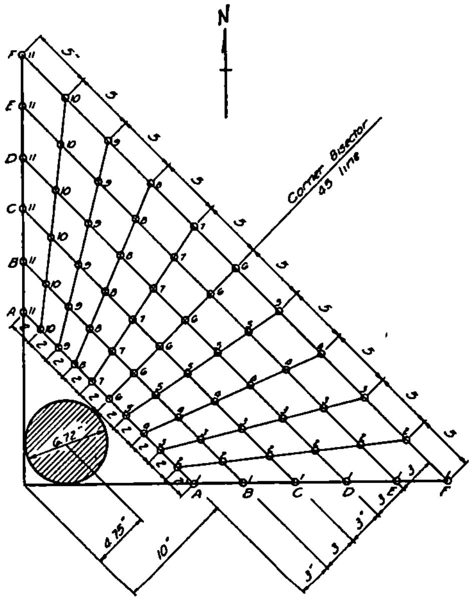


Figure 1. Arrangement of Points for Rosette Strain Readings Slab No 4

rotation of the mirrors caused by the change in slope of the slab as it deflected under load could be eliminated The mirrors of the two extensometers of a pair were arranged to rotate in opposite directions due to surface strains, and since the change in slope caused them to rotate in the same direction, the algebraic average of the scale differences for a pair of extensometers gave the net mirror rotation due to surface strain only The loading arrangement and some of the extensometers may be seen in Figure 2

After the scale differences were taken

[b] This discussion of stresses in slab No 4 is based on data, the greater part of which was obtained by W E Hitchcock (2) and was the basis for his thesis for the degree, Master of Science, granted him by Iowa State College in June 1937

TABLE 2

SAMPLE DATA

Slab No. 4—Diameter of Loading Area, 6.72 in.—Load, 5,000 lb. Line F

Point No.	Measured unit strains x 10^{-6}			Calculated unit strains x 10^{-6}		Direction of maximum principal strain and stress*	Calculated stresses lb. per sq. in.		
	45° to Line F*	90° to Line F*	135° to Line F*	Maximum principal strain	Minimum principal strain		Maximum principal stress	Minimum principal stress	Normal stress
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	− 5.2	+44.0[2]	+45.8	+54.8	−14.7	113.8°	+217	− 4.0	+181
2	− 5.8	+53.0	+41.6	+60.1	−24.2	107.2°	+230	−39.2	+207
3	− 4.6	+54.1	+38.7	+60.0	−25.8	105.2°	+228	−46.1	+209
4	+ 4.1	+65.4	+35.3	+68.0	−28.7	99.5°	+259	−49.8	+252
5	+14.3	+70.2	+33.7	+71.4	−23.1	96.0°	+280	−22.2	+277
6	+23.4	+67.6	+32.8	+68.0	−11.9	90.0°	+277	−21.7	+277
7	+23.5	+69.1	+19.2	+69.1	−26.3	89.0°	+266	−38.4	+266
8	+37.5	+68.4	+ 9.4	+70.6	−23.8	81.5°	+276	−26.0	+269
9	+39.4	+70.3	− 5.8	+74.9	−41.3	78.5°	+275	−96.4	+261
10	−15.9[1]	+51.8	− 6.3	+59.7	−23.8	72.2°	+229	−37.9	+205
11	+43.5	+44.0[2]	− 8.3	+54.7	−19.4	67.9°	+212	−24.3	+179

Plus signs indicate tension, minus signs compression.

* Angles are measured clockwise from Line F (See Fig. 1).

[1] To avoid an obstruction in the line of sight this reading was taken along Line F instead of 45° to it.

[2] Values for points 1 and 11 in this column are extrapolated.

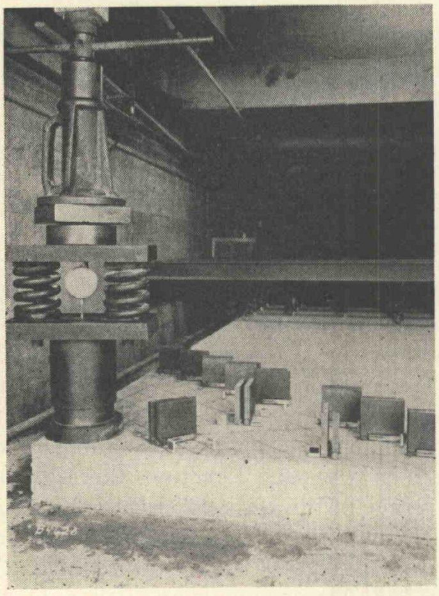


Figure 2. Loading Apparatus and Mirror Extensometers at S. W. Corner of Slab No. 4

and the algebraic average obtained they were converted into unit strains by the formula

$$\delta = \frac{SW}{2GD} \quad (1)$$

in which

δ = unit strain.
S = algebraic average of scale differences.
W = height of the diamond knife edge.
G = gage length.
D = distance from mirror to scale.

Columns 1, 2, and 3 in Table 2 give unit strains at the various points on line F as calculated by this formula. These values are the average obtained by three separate loadings of the slab. Individual readings showed variations of about three to five percent from the mean in regions where the measured unit strains were 20 to 80 millionths. This range included the important strains In some cases the unit strains were below 10 millionths, a few even being less than 1 millionth These values approach the limit of precision of the extensometers which was about $1\frac{1}{2}$ millionths with the lengths of optical levers used Variations of individual readings in this last group ran as high as 20 percent of the mean

Due to a lack of understanding of the stress situation along the edge of the slab, strain readings were taken on only two gage lines at all of the edge points Later it was realized that strain measurements on three gage lines were necessary at edge points as well as interior points The approximate strain on a third gage line was obtained by extrapolating the curves on the strain readings on interior gage lines parallel to this third gage line As a check on this extrapolation the stresses normal to the edge were computed at each edge point and compared with the actual stress normal to the edge which is known to be zero The greatest stress normal to the edge computed in this manner was 17 percent and the average of all values was only 9 percent of the principal stress at the edge

Having these rosette strain measurements, the magnitude and direction of principal strains at each point may be obtained graphically by a method described and explained by Osgood and Sturm (5) A sample of this graphical construction has been carried out for point F-2 and is shown in Figure 3

The principal stresses shown in columns 7 and 8 were obtained from the principal strains by the equations

$$\sigma_{max} = \frac{E}{1-\mu^2}\,(\delta_{max} + \mu\delta_{min}) \quad (2)$$

$$\sigma_{min} = \frac{E}{1-\mu^2}\,(\delta_{min} + \mu\delta_{max}) \quad (3)$$

where σ_{max} and σ_{min} are the maximum and minimum principal stresses respectively, δ_{max} and δ_{min} are the maximum and minimum principal strains Tensile strains are considered positive and compressive strains negative E is the modulus of elasticity and μ is Poisson's Ratio

The normal stresses shown in column 8 were determined from the principal stresses by the equation

$$\sigma_n = \frac{\sigma_{max} + \sigma_{min}}{2} + \frac{\sigma_{max} - \sigma_{min}}{2}\cos 2\theta \quad (4)$$

In this equation σ_n is the normal stress, σ_{max} and σ_{min} the maximum and minimum principal stresses respectively, tensile stresses are considered positive and com-

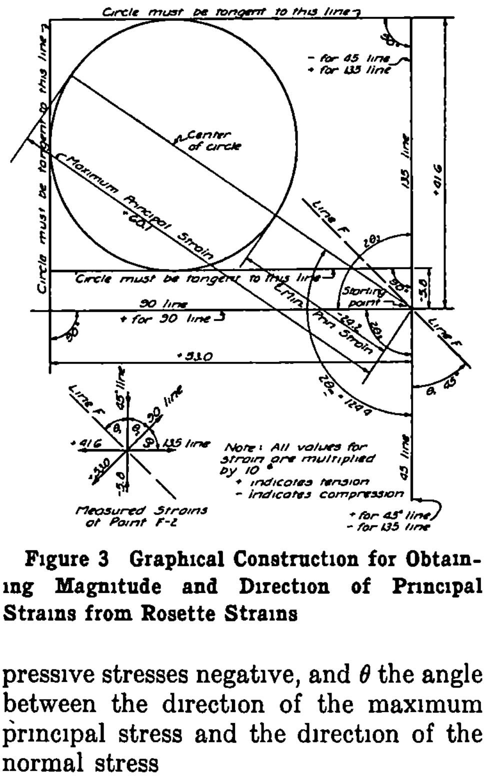


Figure 3 Graphical Construction for Obtaining Magnitude and Direction of Principal Strains from Rosette Strains

pressive stresses negative, and θ the angle between the direction of the maximum principal stress and the direction of the normal stress

These equations may be found in almost any textbook on Mechanics or Theory of Elasticity (7)

The magnitude and direction of the maximum principal stress at each of the 66 points are shown in Figure 4 It is of interest to note that the direction of this principal stress was approximately parallel to the corner bisector for almost all points It may be further noted that there is a comparatively large area in the vicinity of the corner over which the stress was fairly uniform This may also be seen in Figure 5 which is an iso-stress

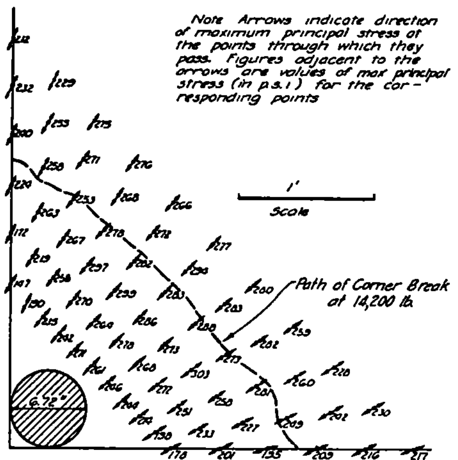


Figure 4 Magnitude and Direction of Maximum Principal Stresses Slab No 4, 5,000-Lb Load

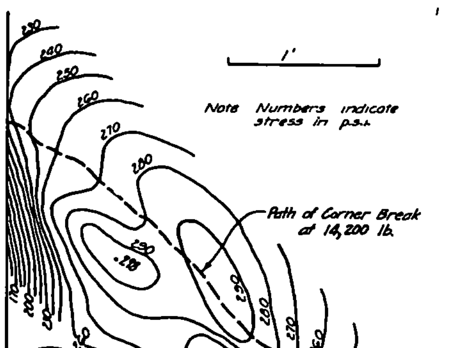


Figure 5 Iso-Stress Diagram for Maximum Principal Stresses, Slab No 4, 5,000-Lb Load

diagram for maximum principal stress, or in other words a stress contour diagram Data for this diagram were obtained by plotting the principal stresses against distance along the radial lines and drawing a smooth curve through the points, which, of course, has the effect of smoothing out the diagram In general, the principal stresses at points along the edges of the slab are about 20 to 40 percent less than the principal stresses along the corner bisector This situation is the reverse of that reported by Murphy (4) whose ana-

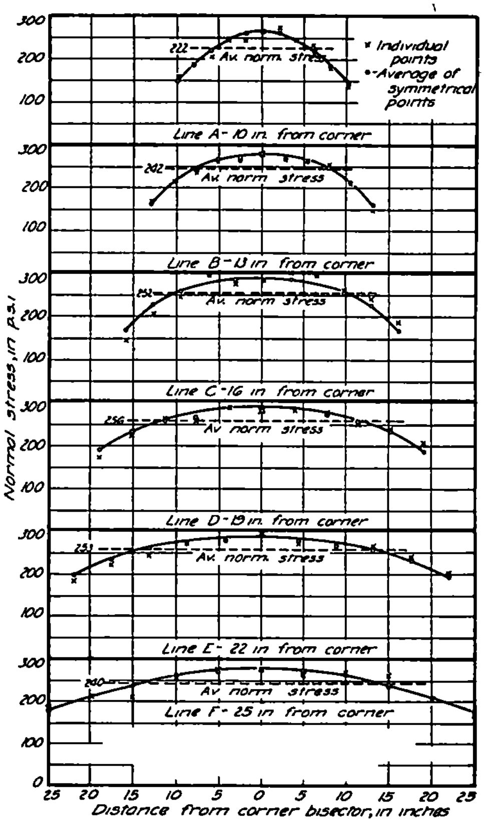


Figure 6 Distribution of Normal Stress Along Lines Perpendicular to Corner Bisector Slab No 4, 5,000-Lb Load

lytical solution yielded greater stresses at the edge than along the bisector

If the normal stresses, such as those in column 8 of Table 2, are plotted over the lines perpendicular to the corner bisector, the curves shown in Figure 6 are obtained The area under these curves divided by the length of the line over which the curve was plotted gives the average normal stress along that line

A planimeter was used to determine the areas

Since the direction of the maximum principal stresses at points on the corner bisector is parallel to the corner bisector due to symmetry about this line, the value of θ in equation 4 is zero Hence, for all points on the bisector, the normal stress is equal to the maximum principal stress

Consider a triangular section of the corner formed by the edges and one of the lines marked by letters (see Fig 1) There is a total bending moment acting on this section caused by the load and the subgrade reaction If this moment is assumed to be uniformly distributed over the width of the section, and a stress calculated from it, that stress will be what has been defined as the average normal stress This procedure was followed by Westergaard (8) in his analytical solution for the stress caused by corner loads and subsequently by Spangler (6) in deriving an equation for minimum probable stress

It can be seen from Figure 6 that the normal stress is not uniformly distributed over lines perpendicular to the corner bisector and that the maximum normal stress is 13 percent to 18 percent greater than the average normal stress depending upon the line considered Therefore, it would seem that if a stress is calculated from a total bending moment by assuming a uniform distribution of moment, approximately 15 percent should be added to that stress to get the maximum value

A comparison of experimental and analytical values of stress along the corner bisector is given in Figure 7 The apparent stress which is the radial unit strain times the modulus of elasticity is from 14 percent greater to 4 percent less than the maximum principal stress The minimum principal stress which is a compressive stress in the top surface gives an indication of the tensile stress which might be expected on the bottom of the slab These tensile stresses in the bottom surface were relatively small in this slab, but some studies by Lightburn (3) indicate that they may be of importance in some cases He investigated small rectangular plaster slabs, 9 in square and ½ in thick, under loads concentrated over a small area and supported on a rubber subgrade With certain loads at the corners of these slabs, he noted that the ma-

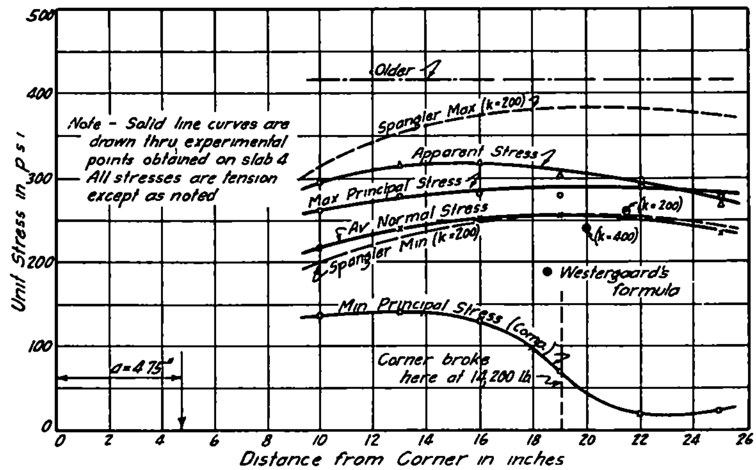


Figure 7 Comparative Values of Stresses Along the Corner Bisector, 5,000-Lb Load

terial failed along the corner bisector by tension in the bottom face before the ordinary corner break occurred

Older's equation (8) for the maximum stress in a slab gives values which are 46 percent greater than the highest experimental value obtained in these tests Assuming a value of 200 lb per sq in per in for the modulus of subgrade reaction, which should seem reasonable in the light of subsequent data, Westergaard's (8)

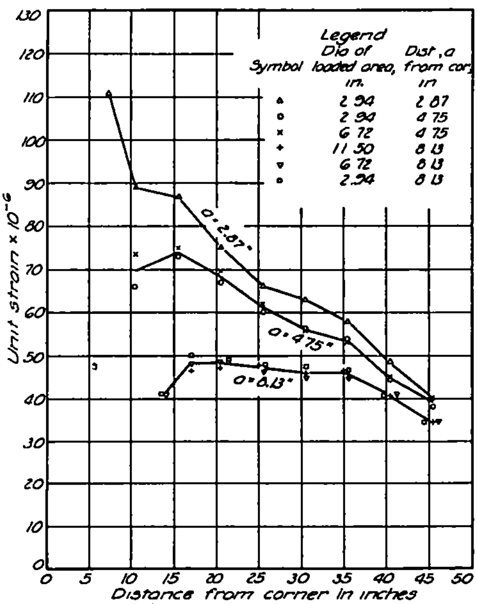


Figure 8 Variation in Radial Strain Along Corner Bisector Due to Size and Location of Loading Area, Slab No 4, 5,000-Lb Load

(9) expression gives a stress which is 10 percent less than the highest experimental value The two expressions for maximum and minimum probable stress suggested by Spangler (6) give results which are 33 percent higher and 14 percent lower, respectively, than the greatest experimental stress

The curves in Figure 8 indicate approximately the effect on stresses of the size and position of the loading area The curves show radial unit strain along the bisector which may be taken as approximately proportional to the stress along the bisector They indicate quite definitely that the stress is a function of the distance from the corner to the centroid of the load and that the stress is more or less independent of the size of the loaded area

After all the strain measurements were completed, a load was applied to the southwest corner of the slab until it broke The path of the corner break, which occurred at a load of 14,200 lb, is shown in Figures 4 and 5

STRESS MEASUREMENTS ON SLAB NO 3

The strain measurements and calculations of principal stresses were carried out on slab No 3 in exactly the same manner as for slab No 4, except that the 50 points at which rosette strains were measured were on only five radial lines passing through the corner These points covered a greater total area than on the later slab As a result of this distribution of points, there were rather wide areas in which no stress values were known and it was difficult to trace the iso-stress lines with certainty However, with certain trends definitely established as the result of the better distribution of points on slab No 4, the data for No 3 were interpreted with more confidence

The directions of the principal stresses were much like those shown for slab No 4, that is, they were approximately parallel to the corner bisector The magnitudes of the principal stresses for several different loading areas and positions of load are shown in the iso-stress diagrams in Figure 9 The oblong shaped loading areas, which contained about 64 sq in. were made similar to the area of contact between an 8 by 40 in pneumatic tire and a flat surface with the tire inflated to 110 lb per sq in and supporting a 6,000 lb load From Figure 9 it may be noted that in this slab as in No 4 there is a considerable area over which the stress does not vary greatly.

The stresses in slabs 3 and 4 caused by the same magnitude and position of load, and with the load distributed over the same area, may be compared by reference to Figure 9 (a) and Figure 5 Although the slabs were the same thickness and approximately the same size, the maximum stress in slab 3 was only about 55 percent of that in slab 4 This difference is probably attributable to the fact that the subgrade under slab 3 was

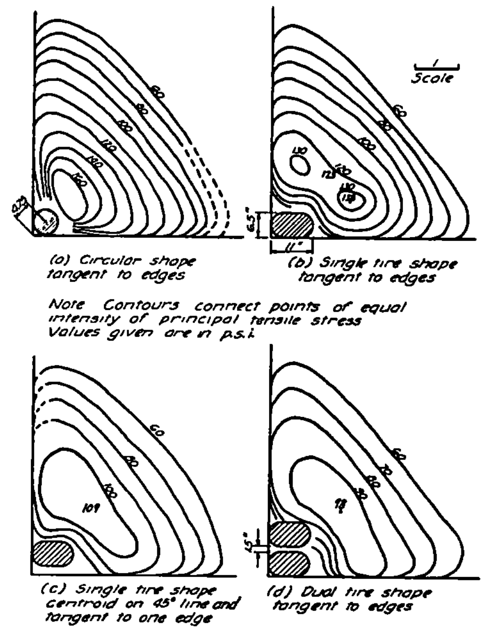


Figure 9 Iso-Stress Diagram for Various Loading Shapes Slab No 3, 5,000-Lb Load

stiffer than that under slab 4, and that the modulus of elasticity of the concrete in slab 3 was less than that in slab 4 It is difficult, however, to thus account for such a wide divergence in stress, since published analyses of stresses (9) (8) (6) indicate that large variations in either or both of these properties cause relatively little variation in stress

SUBGRADE PRESSURES AND SLAB DEFLECTIONS—SLAB NO 4

As previously mentioned, Goldbeck pressure cells were used to measure subgrade pressures Experience on Slab No 3 had shown that a vertical adjustment of the cell was necessary after the cell was in place in the slab A cross-section of this adjustment device and the cell is shown in Figure 10 An outer casing consisting of the cast iron shell A and the $\frac{3}{8}$ in pipe B enclosed the pressure cell C and its connecting pipe This outer casing was held rigidly in place by the bond

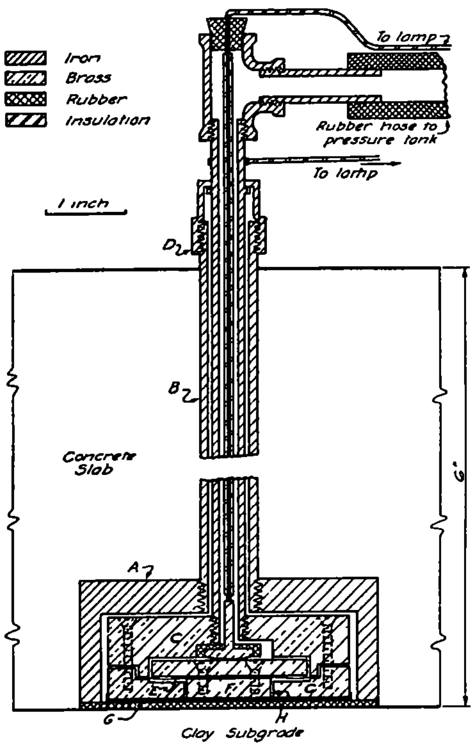


Figure 10 Pressure Cell Adjustment Device

between it and the concrete of the slab The cell could be raised a small amount by simply pulling up on the small inner pipe, and could be moved down and held down by screwing the cap D against the shoulder on the inner pipe Thus an initial pressure reading was assured, and moreover it could be adjusted to any desired value within certain limits A circular piece of rubber $\frac{1}{16}$ in thick was cemented to the bottom of the cast iron shell to keep out dirt, and to protect the brass covering G

When the cells were originally calibrated it was found that they were not consistent and that the electrical contact was broken at a value of internal pressure which was only about one-fourth of the applied external pressure. This was especially objectionable since it was anticipated that the subgrade pressures would be comparatively small, and with one lb. per sq. in. pressure on the slab being registered as only about one-fourth lb. per sq. in. on the manometer there would not be sufficient accuracy.

After considerable study and experimentation it was found that the reason the cells would not calibrate one for one was that the diaphragm E which completely sealed the air pressure from the lower part was tightly pressed against the lower shoulder, and hence could only bend over the narrow width H. Whereas the internal pressure should be acting over an area equal to that of the plunger F, it was acting over this area plus the area of the diaphragm which pulled the lower portion down, breaking the electri-

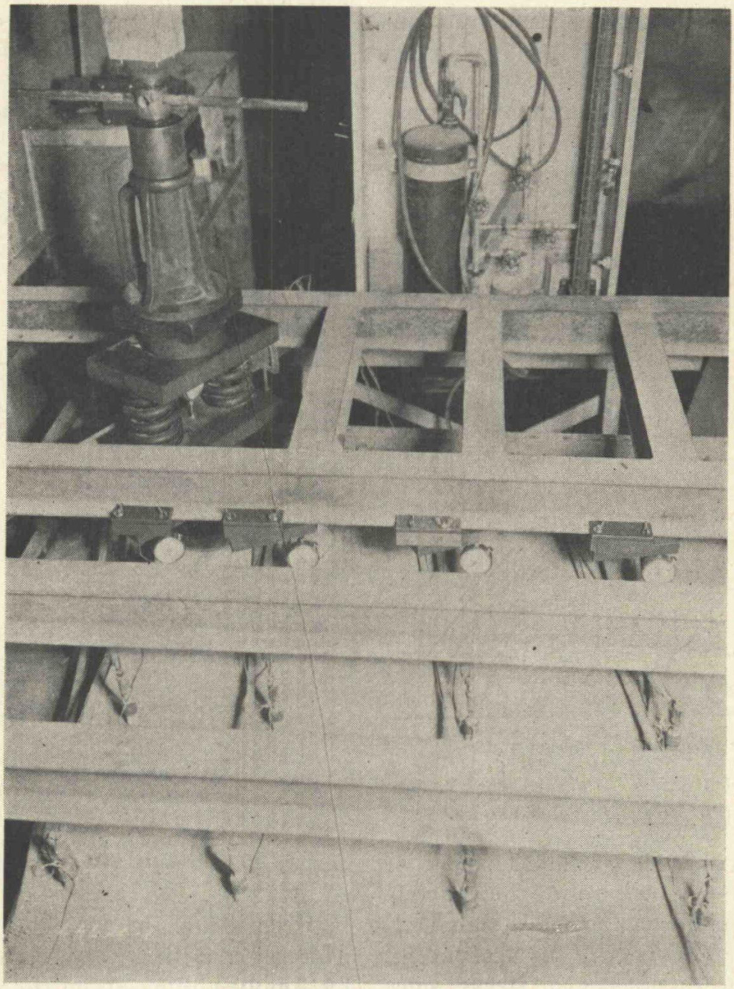


Figure 11. Apparatus for Measuring Subgrade Pressures and Framework Supporting Deflection Dials at N. W. Corner of Slab No. 4.

cal contact long before the internal pressure was equal to the external pressure If the diaphragm had had no rigidity or had been allowed to bend over a greater width it would have been pressed down on the shoulder without carrying the contact with it

In order to eliminate this difficulty, a couple of $\frac{1}{8}$ in holes were punched in the diaphragm E, and the thin brass covering G made air tight The diaphragm E then had equal air pressure on both sides and could bend over its entire width Since the external pressure against the plunger F sealed the bottom of it from the internal pressure, the external pressure and the internal pressure now acted over the same areas, and the cells calibrated one for one

In actual use in the slab a separate hose and electrical circuit were carried from each cell and rigidly fixed to a panel where the electrical and air hose connections were made at the time the cells were read Some idea of the arrangement may be obtained from Figure 11 Air pressure was supplied from the pressure storage tank and the pressure at the time the electrical contact was broken was measured by a mercury manometer calibrated in ounces The location of the cells in the slab is shown in Figure 12. The numbers by the cells will be used subsequently to designate a given position on the slab

Since there was always an initial pressure on the cells, they obviously recorded change in pressure due to the applied load and not absolute pressure A preliminary study was made to determine what effect various values of this initial pressure, when the slab was unloaded, had on the change in subgrade pressure recorded when the slab was loaded

It was found that as the initial pressure was increased, the change in pressure due to a given load was also increased although not in direct proportion This was especially true of the cells located near the corner Therefore, it was decided that the cells should be adjusted foı an initial pressure equal to that caused by the weight of the slab which was approximately 0 5 lb per sq in

With the cells all set at the above initial pressure, the slab was loaded in increments and the pressures recorded at loads of 3,000, 4,000, and 5,000 lb The difference between the reading at a given load and the initial value gave the pressuıe caused by the applied load and will be ıeferred to subsequently as simply the subgrade pressure The tests showed that this subgrade pressure at any given cell

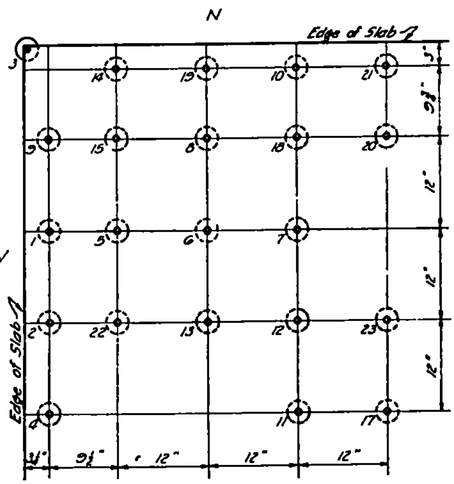


Figure 12 Location of Pressure Cells

varied directly with the applied load The variation in the pressure for different positions in the slab at a given load is shown in Figure 13 An empirical relationship derived from the experimental data is also shown in the figure

$$p=0.0044P_c\, a^{-0.4}\, e^{-0.07r} \quad \ldots \quad (5)$$

in which

p = subgrade pressure.
P_c = load at corner
a = distance from corner to center of loading area
e = base of natural logarithms = 2 718
r = distance from corner to pressure cell

It should not be construed that this equation will give the subgrade pressure on any pavement slab, for it is intended to apply only to the particular slab under investigation here

It can be seen in Figure 13 that for the smaller distances from the corner there are usually two points above the curve and two below it The reason for this is that symmetrically placed cells did not give the same pressure readings, that is, one cell gave high readings and the other one low readings even though they were the same distance from the corner and symmetrically located

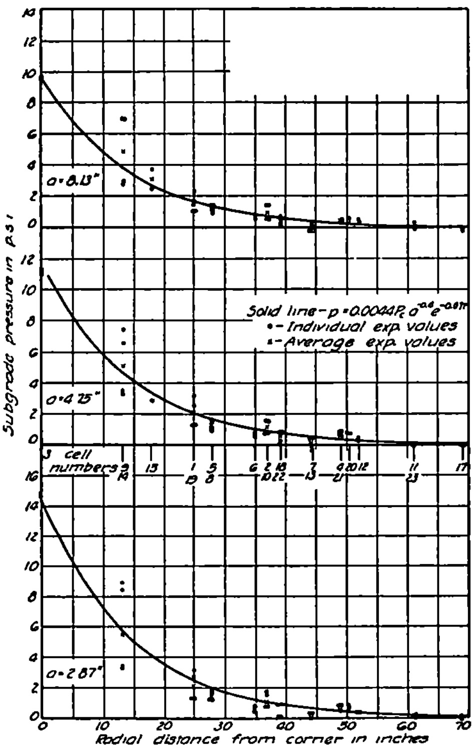


Figure 13 Subgrade Pressures Slab No 4, 5,000-Lb Load

The deflections of the slab at each cell position were measured with 0 0001-in Federal dials, which were attached to a steel framework erected to hold the dials rigidly in position A part of this framework may be seen in Figure 11 It consisted of an 18-in I-beam which spanned the slab and was supported on concrete piers. Several 4-in I-beams were bolted between this beam and the concrete basement wall, and the dials fastened to these beams with a special clamp It was necessary to load the slab twice in order to get a complete set of deflection readings, since only ten dials were available

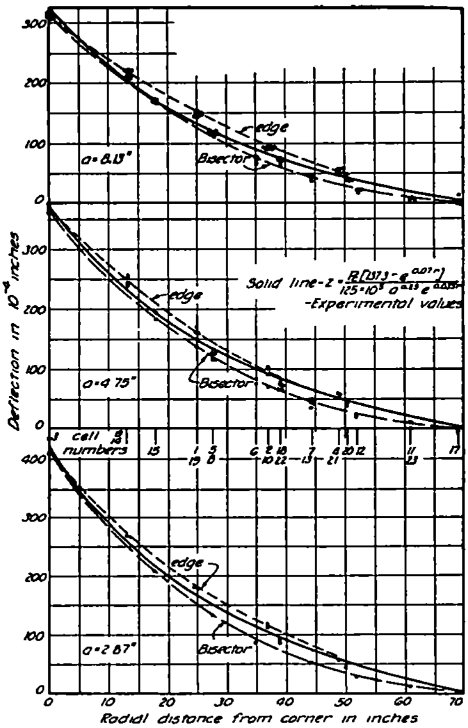


Figure 14 Deflection Curves Slab No 4, 5,000-Lb. Load

As with pressures, it was found that the deflection at a given point varied directly with the applied load The variation in deflection from point to point for a given load is shown in Figure 14 More deflection values are shown than pressure values because deflection readings were taken during the preliminary study of pressures and these have been included. The data show that the deflections along an edge fall on one curve while those along the corner bisector fall on another However, these curves are not far apart, and the deflections for a given magnitude and position of load approximately follow a curve which depends only on the distance from the corner An empirical equation,

$$z = \frac{P_c[137.3 - e^{0.07r}]}{125 \times 10^5 a^{0.25} e^{0.035r}} \quad (6)$$

was derived from the experimental deflection data

The measured deflections show that the slab dishes slightly so that it is concave

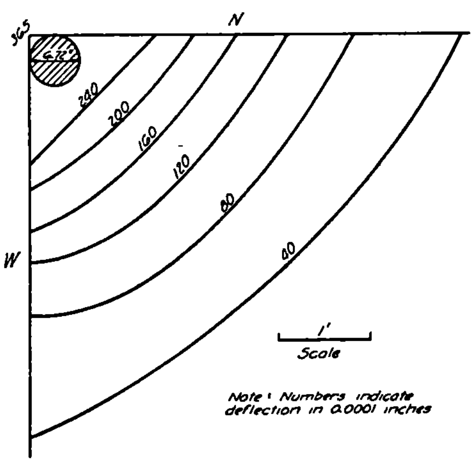


Figure 15 Iso-Deflection Diagram, Slab No 4, 5,000-Lb Load

upward as may be seen in the iso-deflection diagram given in Figure 15 This effect increases as the position of the load is moved away from the corner

Since both the subgrade pressure and the deflection at a given point on the slab varied directly with the load, the ratio of the subgrade pressure to the deflection at a given position is a constant However, the value of this ratio varies with the distance from the corner. This fact is shown in Figure 16 where the curve is the ratio of the empirical expression for pressure to the empirical expression for deflection The ratios of the average of the subgrade pressure readings to the average of the deflection readings at corresponding points give quite a scattered set of values especially at the larger distances from the corner where both the pressure and deflection readings were small The pressure readings in this region were probably less than the precision limits of the pressure cells

Goldbeck's (1) tests on the effect of size of bearing area on the supporting

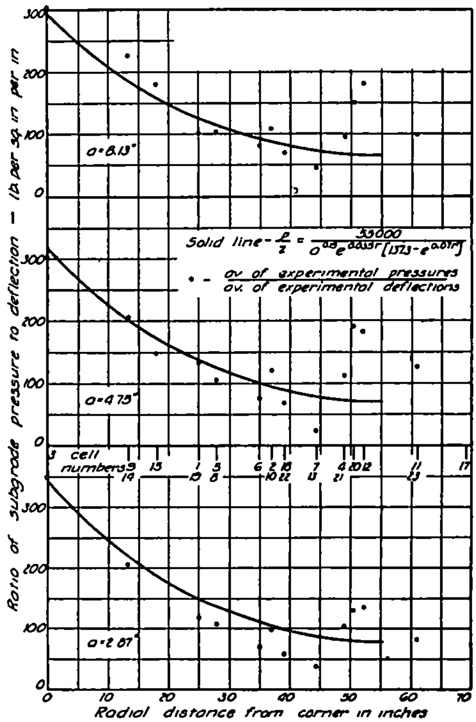
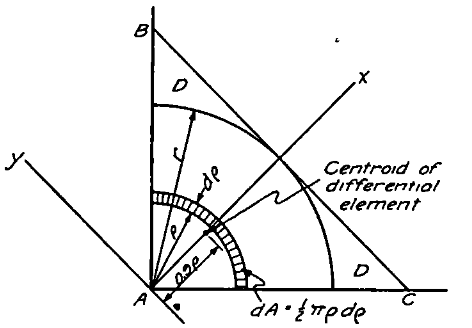


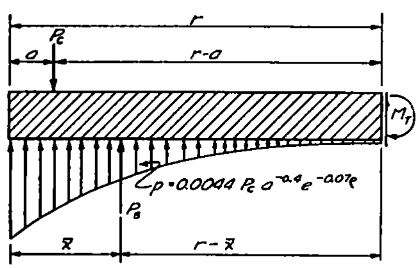
Figure 16 Ratio of Subgrade Pressure to Deflection Slab No 4

value of soils show that the ratio of the pressure to the deflection varies approximately inversely as the square root of the area If the slab is thought of as being broken up into narrow areas perpendicular to the corner bisector, approximately the same relationship exists between these areas and the ratio of subgrade pressure to deflection as Goldbeck observed in his tests Westergaard (8) called this ratio of subgrade pressure to deflection the "modulus of subgrade reaction," and in his analytical solution assumed it to be a constant of the same value for every point within the area considered. Nevertheless, he recognized that it might vary from point to point in the slab, and showed that different values of the ratio in his equations did not materially affect the value of the stress.



Pressure on area, $p = 0.0044 P_c a^{-0.4} e^{-0.07\rho}$

(a) Plan View of Corner



(b) Cross-section Through Corner Bisector

Figure 17

STRESS CALCULATION FROM MEASURED PRESSURE

If the magnitude and position of the centroid of the total subgrade pressure acting on a triangular section of the slab such as the area ABC in (a) Figure 17 could be determined, all of the external forces acting on the section would be known. With the forces known, the external bending moment on the section could be found and the average normal stress over the length BC calculated. In order to get the total subgrade pressure, the character of the pressure variation over the area must be known. The empirical expression for the subgrade pressure gives this variation for this slab.

An attempt was made to integrate the subgrade pressure over a triangular area, but the expression encountered could not be integrated. Therefore, it was decided to integrate over a circular sector and try to determine the error introduced. A conservative estimate indicated that this error was less than one percent.

Referring to Figure 17(a) it can be seen that the total subgrade pressure P_s acting on any given circular sector, may be expressed by

$$P_s = \int p\,dA = 0.0022\pi P_c a^{-0.4}\int_0^r \rho e^{-0.07\rho} d\rho = 0.099 P_c a^{-0.4}[14.3 - 14.3e^{-0.07r} - re^{-0.07r}] \quad (7)$$

To get the position of P_s, its moment with respect to the y-axis may be determined

$$\begin{aligned} M_y &= 0.9\int \rho p\,dA = 0.00198\pi P_c a^{-0.4}\int_0^r \rho^2 e^{-0.07\rho} d\rho \\ &= 0.0888 P_c a^{-0.4}[408 - 408e^{-0.07r} - 28.6re^{-0.07r} - r^2e^{-0.07r}] \quad (8) \end{aligned}$$

The distance from the corner to P_s is

$$\bar{x} = \frac{M_y}{P_s} = \frac{0.897[408 - 408e^{-0.07r} - 28.6re^{-0.07r} - r^2e^{-0.07r}]}{14.3 - 14.3e^{-0.07r} - re^{-0.07r}} \quad (9)$$

From the free-body diagram in Figure 17(b) the total external bending moment, M_T, may be obtained

$$M_T = P_c(r - a) - P_s(r - \bar{x}) \quad (10)$$

Substituting the values of P_s and $\bar{x}$ from equations 7 and 9 gives

$$M_T = P_c(r-a) + 0.099P_c a^{-0.4}[367 - 14.3r - 367e^{-0.07r} - 11.4re^{-0.07r} + 0.1r^2e^{-0.07r}] \quad (11)$$

If the total moment M_T is divided by 2r, the width of the section BC, the average moment per unit of width, M_u, is obtained

$$M_u = \frac{P_c}{2}\left(1 - \frac{a}{r}\right) + 0.0495P_c a^{-0.4}\left[\frac{367}{r} - 14.3 - \frac{367}{r}e^{-0.07r} - 11.4e^{-0.07r} + 0.1re^{-0.07r}\right] \quad (12)$$

The average normal stress may now be calculated from the flexure formula,

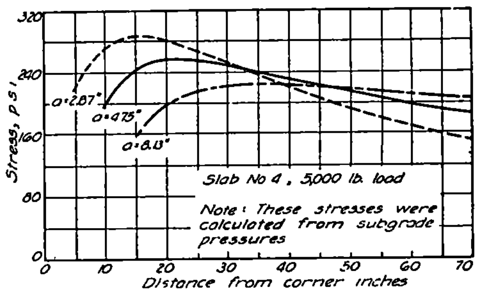
$$\sigma_n = \frac{6M_u}{h^2} \quad (13)$$

Substituting the value of M_u from equation 12,

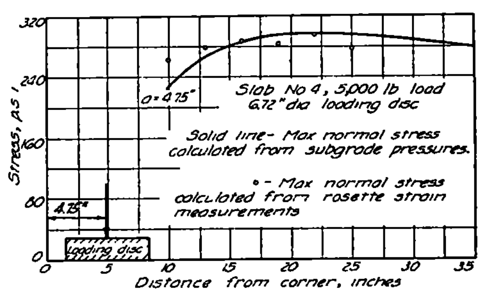
$$\sigma_n = \frac{3P_c}{h^2}\left(1 - \frac{a}{r}\right) + \frac{0.296P_c}{h^2a^{0.4}}\left[\frac{367}{r} - 14.3 - \frac{367}{r}e^{-0.07r} - 11.4e^{-0.07r} + 0.1re^{-0.07r}\right] \quad (14)$$

The above expression for σ_n is an unwieldy one, and can best be interpreted from a graph of the equation The curves showing how this average normal stress, σ_n, varies with the distance from the corner and also with the distance, a, are given in Figure 18 (a) It is interesting to note that as the distance, a, increases, the curves flatten out, and for a = 8.13 in there is a large distance over which the stress is almost constant

It may be recalled from the previous discussion on average normal stress that to obtain the maximum principal stress from the average normal stress approximately 15 percent should be added to the latter Figure 18 (b) shows the average normal stress curve for a = 4.75 in with 15 percent added to it and represents approximately the maximum principal stress The plotted points are the maximum principal stress as determined from the rosette strain readings on another corner of the same slab The calculations from the empirical equation for pressure give stresses which are fairly consistent with the principal stresses obtained from the rosette strain readings, and it seems should lend a certain amount of confidence to the subgrade pressure measurements



(a) Average Normal Stress along Corner Bisector



(b) Maximum Normal Stress along Corner Bisector

Figure 18

CONCLUSIONS

The foregoing data and calculations which are applicable to the two plain concrete slabs of uniform thickness, supported on a clay subgrade, and loaded at a corner, lead to the following conclusions in regard to these slabs

1 Normal stresses were not uniformly distributed over lines perpendicular to the corner bisector, but were 33 to 45 percent less at the edges than at the bisector Principal stresses at points on these lines were 20 to 40 percent less at the edges than at the bisector

2 There was a comparatively large area in the region of the corner over which the magnitude of the principal stress did not vary greatly

3 The maximum stress in the slab was not materially affected by the area over which the load was distributed, but increased as the centroid of the loaded area approached the corner

5 Apparent stresses were approximately 15 percent greater than the principal stresses in the vicinity of the maximum stress in the slab

6 The ratio of subgrade reaction pressure to deflection was a constant at a given point in the slab, but it varied with the distance of the point from the corner. However, this fact does not seem to affect the stresses computed from the formulas in which it is assumed that the ratio is a constant of the same value at all points

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DISCUSSION ON CONCRETE PAVEMENT STRESSES

DR RUDOLF K BERNHARD, *Consulting Engineer, Baldwin-Southwark Corporation, Philadelphia* This discussion relates to stresses in concrete pavement slabs induced by *dynamic* loads in contrast to the discussion by Spangler and Lightburn of stresses induced by *static* loads The main purpose of the test reported herein was to determine whether the new method with "induced" vibrations, which has been successfully used for soil tests, can be applied to the investigation of stresses in concrete slabs

Tests were made on a concrete highway which served to answer affirmatively the following questions

1 Will the test indicate the relative rigidities of concrete pavement slabs of different thicknesses laid on a uniform subgrade?

2 Does the area over which the force is applied have any influence in the case of a concrete slab?

A description of the technic of the

dynamic tests was given at the December, 1936, meeting of the Highway Research Board.[1]

The fundamental principle is very simple. A highway under investigation is loaded in various places by alternating forces having a sine form. Frequency and size of these forces can be changed. Hence the slab must vibrate with forced and damped oscillations.

The apparatus for exciting these alternating forces, the "oscillator" is a relatively small machine, which has two discs eccentrically supported. The discs revolve in opposite directions and are rotated by an electric motor. Changing the eccentricity of these discs will change the amount of the centrifugal forces; changing the motor input will change the speed of revolutions, and hence the frequency of the exciting forces. All horizontal forces are neutralized in the body of the machine by the inverse rotation of the two discs. Two external vertical forces alternating in a pure sine form remain. Figure 1 is a picture of the apparatus.

The propagation speed, or phase difference of these induced oscillations, can be determined by measuring with seismometers the phase of two corresponding maxima or minima. The time which is required by the wave to move from a point on or near the oscillator to another point causes a phase difference in both sine curves plotted by the seismometers. By moving the seismometers from point to point, it is possible to record closed profiles or contours with no danger of omitting salient points.

The instruments to record the vibrations consist of two pick-up units to record vertical vibration-amplitudes. Based on the seismic principle their natural frequency is low, approximately only 4 cycles per sec., their damping is carefully controlled to avoid any phase distortion. The recorder is a special com-

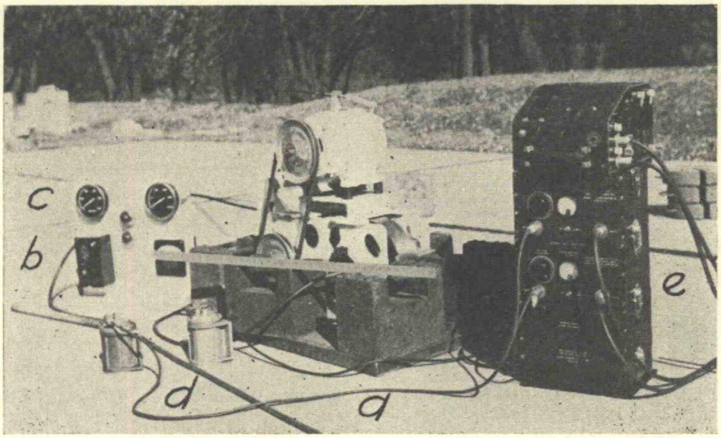

Figure 1

a. Oscillator to Excite Vibrations
b. Switch Board for Oscillator
c. Tachometers to Measure Frequency of Induced Vibrations
d. Pick-Up Units to Measure Vibrations
e. Amplifier and Recorder for Pick-Up Units

[1] *Proceedings,* Highway Research Board, Vol. 16, p. 288 (1936).

pact type of oscillograph containing two elements with a natural frequency of 2500 cycles per sec An amplifier unit enlarged up to 1300 times and transformed the voltage induced by the two pick-ups from speed to amplitude, i e, independent of the frequency within the required range Photographic records can be obtained on a standard 16 millimeter film with a propagation speed of 6¼ and 12½ in per sec respectively Hence, two amplitude measurements can be recorded simultaneously

TESTS

The total width of the experimental highway was 20 ft The dimensions between the expansion joints and outside edge of each concrete slab were 10 × 20 ft The thickness of the slabs varied from 6 to 9 in Three different series of investigations were carried out

First Series The oscillator was placed near the outer edge of one slab, in order to have equal distance from both longitudinal and transfer joints The transfer joint had a **Y**-shape, the longitudinal joint a **U**-shape with steel reinforcement The oscillator excited centrifugal forces of ±600 lb (10° eccentricity) at a frequency of 27 cycles per sec, and ±300 lb at 22 cycles per sec The first pick-up was moved from a point near the oscillator in steps to a maximum distance of 100 ft The maximum sensitivity was, however, not reached in this distance The second pick-up remained always at the first point as reference and control

The density of the soil has not been substantially improved by the concrete pavement Both slab and soil vibrate almost as a unit and no sliding motion will occur between the adjacent surface Hence, the slab should be made only strong enough to resist local stresses by outside loads (vehicles) The relative phase speed in slabs of different thicknesses and the subsoil thus forms a convenient means, in certain cases, to predetermine the required thickness of pavements

The high amplitudes along the free longitudinal edges of the slabs produce the so-called "flutter effect," which indicates the importance of concrete and steel reinforcement along these edges

Second Series The oscillator was placed in the center of a 9 in thick slab, and the seismographs moved systematically step by step over the complete surface of this slab Figure 2 represents the maximum half-amplitudes in two ordi-

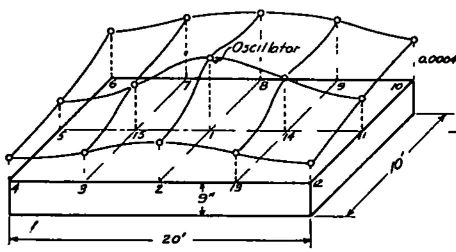

Figure 2 Maximum Deformation of 9-In Concrete Slab Under Dynamic Load in Center of Plate

Maximum Exciting Force, 600 Lb at 27 Cycles Per Sec

Maximum Half-Amplitude, 0 00043 In

Maximum Stress at Point I, 7 Lb Per Sq In

nates Again the continuity of the waves near the adjacent joints and the relatively higher amplitudes at the free edges can be clearly recognized The maximum stress (7 lb per sq in) occurs under the exciting force, causing a steep incline of amplitudes in both ordinates (maximum amplitude 0 00086 in)

Third Series The same investigation as described under Series Two was repeated on a 6-in slab Figure 3 shows the maximum half-amplitude in two ordinates The same phenomena as in Series Two can be observed The maximum amplitude has increased to 0 0022 in (1 2 5) The peak stress has been determined at 12 lb per sq in

The pronounced hump which is also clearly to be seen in the two figures has increased in Figure 3 which indicates obviously the varying rigidity of the slabs or in other words the influence of the thickness of the slabs, including the effect of the subsoil

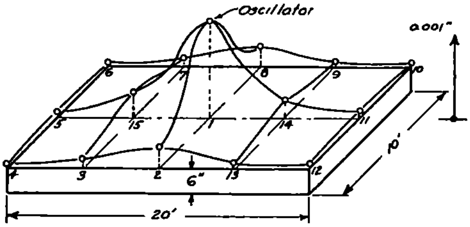

Figure 3 Maximum Deformation of 6-In Concrete Slab Under Dynamic Load in Center of Plate

Maximum Exciting Force, 600 Lb at 27 Cycles Per Sec

Maximum Half-Amplitude, 0 0011 In

Maximum Stress at Point I, 12 lb Per Sq In.

Furthermore, the conclusion might be drawn that a broader distribution of the applied load will flatten substantially the height of this hump and reduce the local stresses

MR R D BRADBURY, *Wire Reinforcement Institute* The results disclosed by this experimental investigation indicate that the actual maximum intensity of stress is slightly greater than that computed by the formula for corner loading in accordance with Westergaard's theoretical analysis Apparently this discrepancy is largely attributable to the fact that the intensity of observed stress is not uniformly distributed on a straight-line section across the slab corner as is assumed by Westergaard But it is possible that, under actual road conditions, other factors may be involved, such as the subgrade, which might not always act in the ideal manner assumed in the theoretical analysis

For example, Spangler,[1] in his preliminary investigations, found evidence of a decrease in effective subgrade pressure after a corner load had been repeated several times As stated by Spangler, this suggested the possibility of a slight permanent compaction of the subgrade in the vicinity of the corner as a result of repeated loading If such a condition tends to develop in practice, then it is reasonable to conclude that the virtual subgrade modulus which is effective for the corner segment as a whole might be materially less than the normal modulus as based upon continuous initial contact between the slab and the subgrade Of course, any attempt to evaluate a condition of this kind necessarily requires a purely arbitrary assumption as to the amount of reduction to be applied to the normal subgrade modulus But the influence of the subgrade is such that a rather wide range in the value of the subgrade modulus has a relatively minor effect upon the intensity of computed stress

According to Westergaard's analysis the value of the radius of relative stiffness of slab to subgrade, l, varies inversely as the fourth root of the subgrade modulus, k If, in arbitrarily fixing the effective value of k for the corner, one assumes for example that its value is only $\frac{1}{4}$ of the normal modulus, then the stiffness radius applicable to the case of corner loading would become $l\sqrt[4]{4}=l\sqrt{2}$

The Westergaard formula[2] for maximum stress under corner loading is,

$$S=\frac{3P}{h^2}\left[1-\left(\frac{a\sqrt{2}}{l}\right)^{0.6}\right]$$

If, according to the above assumption, $l\sqrt{2}$ is substituted for l, one would obtain a modified corner formula,

$$S=\frac{3P}{h^2}\left[1-\left(\frac{a}{l}\right)^{0.6}\right]$$

[1] See *Proceedings*, Highway Research Board, Vol 15, p 122

[2] See *Proceedings*, Highway Research Board, Vol 5, p 90

According to the paper, the general test conditions applying to Slab No 4 were,

$$
\begin{aligned}
E &= 4{,}000{,}000 \text{ lb per sq in} \\
u &= 0.25 \\
a &= 3.36 \text{ in} \\
P &= 5{,}000 \text{ lb} \\
k &= 200 \text{ lb per in}^3 \\
h &= 6 \text{ in}
\end{aligned}
$$

from which the computed radius is $l = 24.9$ in Applying the above modified formula, the computed maximum stress is,

$$S = \frac{3 \times 5000}{6 \times 6}\left[1 - \left(\frac{3.36}{24.9}\right)^{0.6}\right] = 292 \text{ lb per sq in}$$

which corresponds very closely with the observed maximum principal stress of 294 lb per sq in on the corner bisector as shown in the authors' Figure 4

Subsequent tests with different conditions may, of course, fail to exhibit as close a check with the modified formula as is obtained for this specific case But, until valid relationships are revealed by further investigations, it would seem not only desirable but entirely permissible, for practical purposes of stress computation, to make use of some such modification of the Westergaard formula—a formula which is both plausible and consistent in general form and which has the commendable feature of algebraic simplicity

THE STRUCTURAL DESIGN OF FLEXIBLE PIPE CULVERTS

By M G Spangler

Associate Structural Engineer, Iowa Engineering Experiment Station

(In Abstract*)

There is no rational method available to the engineering profession for predicting the structural performance of flexible type culvert pipe in advance of installation This paper is a progress report on a research project being conducted by the Iowa Engineering Experiment Station in cooperation with the U S Bureau of Public Roads to study structural performance and to develop a rational theory of design

In the case of flexible pipes, the pipe itself has relatively little inherent strength, and a large part of its ability to support vertical load must be derived from the passive pressures induced as the sides move outward against the earth Analysis of structural behavior, then, must take into account the earth at the sides as an integral part of the structure

Flexible pipe sections of various diameters were loaded in the laboratory at diametrically opposite points and the measured deflections were compared with those calculated by the thin ring elastic theory Results were found close enough to justify the conclusion that even though the deflections and accompanying changes in radius of curvature are relatively large, the elastic theory is applicable to corrugated metal pipes under two point loading within a tolerance probably no greater than that occasioned by variations in the modulus of elasticity of the metal, variations in gage thickness, depth and spacing of corrugations, and other variables inherent in the manufacture of this type of conduit. It seems tenable to assume that the theory will also apply in the case of a corrugated culvert pipe installed under an embankment, since the external pressures on the pipe in the field will be more nearly uniformly distributed around the pipe than in the laboratory An investigation of the structural performance of corrugated metal pipe culverts, therefore, becomes mainly a study of the laws of magnitude and distribution of the loads and pressures to which they are subjected in service. The hypothesis of fill loads on a flexible culvert pipe under an embankment may be summarized as follows

1 The vertical load may be determined by Marston's theory of loads on conduits, and is distributed approximately uniformly over the breadth of the pipe

2 The vertical reaction is equal to the vertical load and is distributed approximately uniformly over the width of bedding of the pipe

3 The horizontal pressures on the sides of the pipe are distributed parabolically over the middle 100 degrees of the pipe and the maximum unit pressure is equal to the modulus of passive resistance of the filling material multiplied by one-half the horizontal deflection of the pipe

Having set up such an hypothesis, it has been possible to develop mathematical expressions for the moments, thrusts, shears and deflections of a pipe in terms of the properties of the pipe and the earth of which the embankment is constructed Finally an equation has been tentatively adopted for use in the design of flexible culvert pipes when the conditions of installation are sufficiently well known to justify the calculation of the vertical load on the pipe by means of Marston's conduit load theory, and

* This report has been published in full in *Public Roads*, Vol. 18 No 12 February, 1938

when the passive resistance of the filling material is known or can be estimated within a reasonable tolerance This equation is expressed as follows

$$\Delta X = \frac{KW_c r^3}{EI + 0\,061 er^4}$$

in which

ΔX = horizontal deflection of flexible culvert pipe

$K = 0\,500 \sin\alpha - 0\,082 \sin^2\alpha + 0\,080 \frac{\alpha}{\sin\alpha} - 0\,160 \sin\alpha\ (\pi - \alpha) - 0\,040 \frac{\sin 2\alpha}{\sin\alpha} + 0\,318 \cos\alpha - 0\,208$

W_c = vertical load per unit of length of pipe

r = mean radius of pipe

E = modulus of elasticity of pipe metal

I = moment of inertia per unit of length of cross section of pipe wall

e = modulus of passive resistance of the enveloping earth

α = one half the bedding angle

In order to test the applicability of this design formula, an extensive experimental program has been conducted in which four corrugated metal pipe culverts have been installed at the Experiment Station field laboratory at Ames and loaded with a clay embankment 15 feet high above the top of the culverts The pipes for these experiments and the laboratory studies previously referred to were of four different diameters and U S gage thicknesses namely, 36-in 16-gage, 42-in 14-gage, 48-in 14-gage, and 60-in 12-gage Light gages were chosen so that the diameter changes under the fill loads would be relatively high The embankment was 17 feet wide on top with side slopes about 1 2 on 1 The site for the experimental embankment was the bottom of an old gravel pit The bedding for the culverts was prepared by cutting out a trench with circular shaped cross section and a radius 2 in greater than the pipe, refilled with sand which was struck off with a template of the same radius as the pipe When the pipes were laid they were in contact with the shaped sand bedding for the bottom 90 degrees of the circumference and projected above the subgrade a distance equal to 0 85 of their diameter

Measurements and observations made during the experiments were directed toward three principal objectives First, the settlement of various elements of the pipes and adjacent embankment material was observed in order that the settlement ratio of Marston's theory could be calculated Also the unit weight of the fill material was measured, and all other data necessary to calculate the load on the culverts according to this theory. Second, the distribution of vertical load, vertical reaction and horizontal pressures were measured in order to check the hypothetical distributions and to determine the value of the modulus of passive resistance of the clay filling materials both in the tamped and untamped condition Third, the vertical and horizontal deflections of the pipes were measured for comparison with the hypothetical deflections computed by the design equation

The distribution of the vertical and horizontal pressures on the pipes was measured by means of stainless steel friction ribbons During the loading period the settlements, deflections and ribbon pulls were observed at least once for every one foot increment of fill

It appears from examination of the deflection and foundation settlement curves that the maximum load on the culverts was reached within a very short time after the fill was completed, probably within less than a week

The relationship between the lateral pressure on the pipes and the horizontal movement of the sides of the pipes was determined, that is, the modulus of passive resistance of the fill material For the untamped clay it was 13 4 lb per sq in per in movement and for the tamped filling material 27 0 lb per sq in per in It appears, therefore, that tamping the side fills practically doubled their capacity to assist the pipes in carrying the vertical earth load

Having determined the modulus of passive resistance of the side fill materials, and knowing the physical properties of the pipes, the deflection of the pipes under the calculated load may be determined by the tentative design formula

An interesting phenomenon is the fact that the pipes continue to deflect slowly long after the fill is completed and the maximum vertical load attained More than a year after completion of this fill deflection continued, the average amount of this lag being 0 25 per cent of the nominal diameter with the tamped side fills and 0 38 per cent for the pipes with untamped side fills.

In these experiments the straight-line increase of both deflections and side pressures as the fill was constructed justifies the use of a constant ratio between deflection and pressure in this ordinary method of construction The effect of vertical pressure on the ratio of deflection to side pressure is not revealed in these studies, however, so that when a flexible pipe is "strutted" before the fill is placed and the strutts afterwards removed, a different situation is presented Much remains to be learned, furthermore, regarding characteristics of the side fill materials and their effect on the modulus of passive resistance Study is also needed of other bedding conditions

DISCUSSION OF FLEXIBLE PIPE CULVERTS

Mr George E Shafer, *Armco Culvert Manufacturers Association:* We have followed with a great deal of interest Dean Marston's very complete investigation of loads on closed conduits and the design of rigid pipe We naturally have an even greater interest in Mr Spangler's continuation of this excellent type of work and the effect it will have on the design and acceptance of corrugated metal pipe There is a great contrast between Mr Spangler's comprehensive understanding of flexible pipe and the original ideas of the inventor, who 41 years ago made the first corrugated metal pipe with the corrugations running parallel with the pipe instead of circumferentially The correct way to run the corrugations was soon found and a new industry started to grow, slowly at first because many questioned the structural strength of a pipe with walls so thin

Records show a continual battle on the part of producers to prove to the public that the pipe was strong enough even though no one could figure out why They resorted to borrowing elephants from side shows to stand on corrugated pipe at fair exhibits, or threshing machines to run over the pipe

Just after Professor Talbot published his classic report on the design of Cast Iron and Concrete Pipe, he was asked to test corrugated metal pipe to see if it was as strong as the rigid pipe, hoping to find a reason for its ability to stand up under high fills These and other tests were made, not to determine the correct gage of metal to use under certain height fills, but to answer the question—why does corrugated metal pipe develop so much field strength?

What producers thought was the proper gage was determined by experience just like bridges were designed prior to 1840 when truss analysis was first introduced It was not until 1924 when Mr Lacher, Managing Editor of *Railway Engineering and Maintenance,* stated that railroads were beginning to recognize the merits of corrugated metal pipe and that a scientific analysis of strength would be helpful, that the subject of analysis received much serious consideration

The exact load on a flexible pipe from a fill was unknown until determined in 1925 by the A R E A 's Farina test Dean Marston soon followed with a complete mathematical theory, presented to this group in 1929

Two independent research groups have tried to design corrugated metal pipe by mechanically loading the pipe to failure using the distribution of pressure determined from field tests

The producers of one group have attempted to check their gage table by developing an empirical equation for the deflection of the pipe Here you have heard the first systematic attempt to design rationally a commonly used structure, the analysis of which has bothered many—not because of the structure itself but because of the lateral pressure developed against the side of the pipe, which depends upon so many variables Mr Spangler's approach is logical because the design must be based upon deflection, the real measure of failure

Deflection being so important we have naturally observed many installations of various fill heights over a long period of time Some data on the question of "deferred" deflection, mentioned by Mr Spangler, may be of interest here.

Unless a corrugated metal pipe does deflect, it is not functioning correctly or utilizing all the available natural resources The deflection must and does eventually cease, but at what rate does it proceed? Several outstanding installations will serve to illustrate this point and answer the question

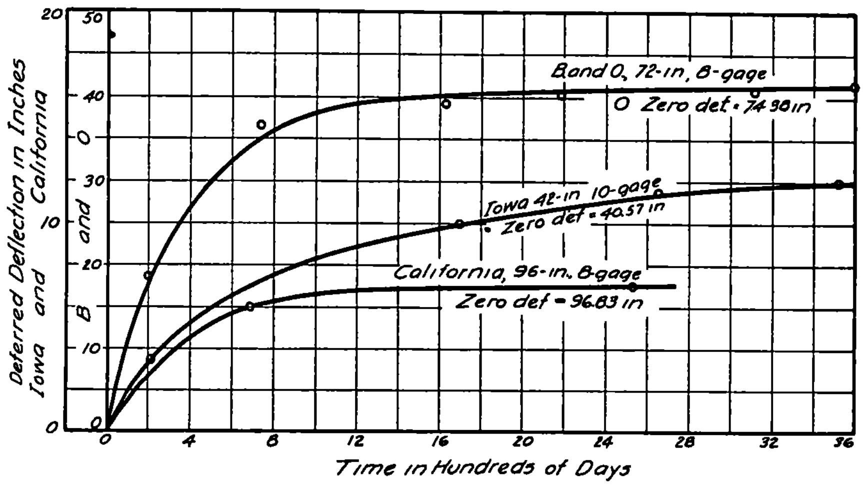

Figure 1

Figure 1 shows the deferred deflection plotted against time for three installations where accurate measurements have been recorded for from 7 to 10 years The upper curve is for a 72-in 8-gage pipe installed under 3 ft of cover on the B & O R R A huge hot metal ladle (Fig 2) weighing 343 tons passes over this culvert several times each day This ladle produces the heaviest known axle load in regular railroad use today The bottom curve of Figure 1 is for a 96-in 8-gage pipe under 25 ft fill on the Calaveras Branch of the Southern Pacific Railroad (Fig 3) Both these installations were strutted; that is, the vertical diameter was elongated while the fill was being made. The side fills were compacted but not especially tamped.

The center curve is for a 42-in. 10-gage pipe installed at Ames, Iowa under a 15 ft. untamped fill (Fig. 4). The fact that the Ames culvert was not strutted and the side fills not tamped probably accounts for the different shape of the curve.

These data which are typical, show how the pipe deflects rather rapidly at first and in so doing builds up sufficient side support to counterbalance its part of the load, then the deflections slow up

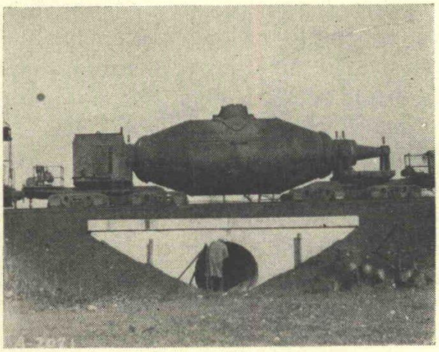

Figure 2. A hot metal ladle weighing 343 tons passes over this culvert several times a day

and eventually cease. The thousands of existing installations are proof that the deflection does cease, but at what rate is of interest.

If the inherent strength of the pipe is more than the load on the pipe, the deflection may cease very soon, at least before final settlement of the fill alongside the pipe. If the inherent strength is less than the load, deflection may continue as long as there is settlement in the fill and that, of course, depends upon the type of embankment material and how placed, etc.

Since deflection is so important on this type of construction, as brought out so clearly by Mr. Spangler, the next desire is to know how far the pipe can deflect, in percentage of the diameter, before collapse will occur. From field studies of old existing installations made of gages far lighter than are good practice today, this point of pending failure has been

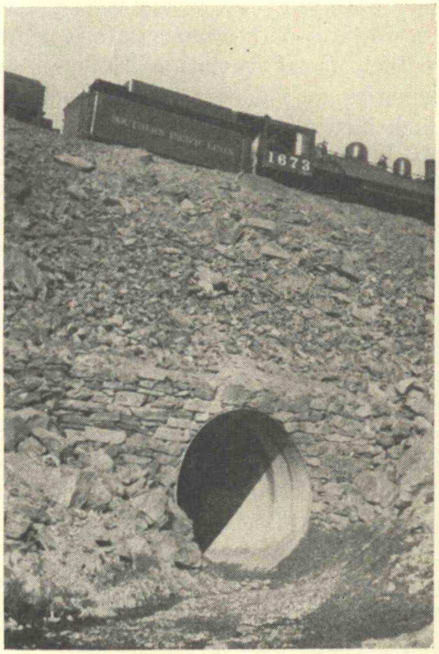

Figure 3. A 96-in. 8-gage culvert 100 ft. long

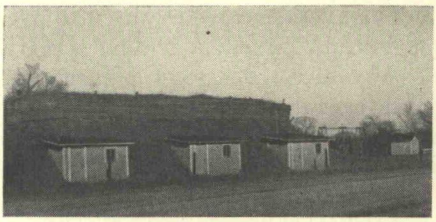

Figure 4. An experimental installation of 42-in. 8-gage pipe under 15 ft. untamped fill at Ames, Iowa.

fixed at 20 per cent. That is a 60-in. pipe can deflect 12 in. before failure. Most designs should be made on the basis of 5 per cent deflection when the fill is practically consolidated, giving a factor of safety of from 3 to 4.

EXPERIMENTAL DETERMINATION OF THE VARIATION IN PRESSURE INTENSITY OVER THE CONTACT AREA OF TIRES

By L W Teller and James A Buchanan
U S Bureau of Public Roads

(In Abstract*)

In order to obtain basic data concerning the actual variations in pressure intensities in the contact area between vehicle tires and pavement surfaces, the Bureau of Public Roads has recently developed a new method for determining such unit pressures and has applied it to two tires representing extreme conditions of cushioning—a solid rubber tire and a thin-walled, low-pressure pneumatic tire (airplane type), each under a 4,000-lb load Both had smooth treads

Each tire was loaded in a universal testing machine, a smoothly-ground steel bearing plate being used to receive the reaction On this bearing plate a 5/16-in wide friction bar of polished brass and suitable filler plates of equal thickness were placed. The tire reaction was transmitted to this built-up plane surface through a sheet of thin steel shim stock which provided a constant friction surface to the upper face of the friction bar The average force required to overcome initial friction in sliding the friction bar between the steel shim and the ground bearing plate was determined for successive parallel positions 5/16-in apart transversely until the entire area of contact had been covered By allocating the total test load to the various elemental 5/16-in wide longitudinal contact strips in proportion to the ratio of frictional force required for each strip to the summation of all frictional forces, a first differentiation, in effect, of the total load on the contact area was accomplished

A second differentiation which apportioned the load applied to each elemental strip according to average differences in initial frictional force required for various lengths of the friction bar covered by the tire, a stationary dummy bar being placed over the remaining portions of the elemental strip, was then accomplished in a similar manner The total load being thus broken down into individual loads on elemental areas, the unit pressure at the center of each elemental area was considered to be equal to the average pressure over that area Such points of unit pressure were then respectively plotted on an outline tracing of the gross contact area, and lines of equal pressure intensity were drawn

The tests show that the maximum load intensities occur in two transversely symmetrical zones rather than at the geometric centers of the areas Analysis of the data suggests the possibility that an ellipsoidal distribution, in which the pressure intensity varies as the ordinates to a semi-ellipsoidal surface of definite proportions, may be a conventionalization which more nearly represents the actual pressure intensity distribution than does the assumption of a uniformly loaded circular area frequently used

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DISCUSSION ON TIRE PRESSURE INTENSITY

MR G M SPROWLS, *Goodyear Tire and Rubber Company*: We have also done some work on the intensity of pressure distribution on pneumatic tires While we have used a different method of making measurements, the type of curves secured from our data follows the same general trend as just shown by Teller and Buchanan Our work has been confined largely to standard truck and passenger tires with tread designs It is to be expected that the intensity of pressure under such conditions would be greater than with a smooth tread tire, and a rather thin tread at that, mounted on a carcass more flexible due to a fewer number of plies We have found relatively higher intensities of pressure than indicated by Teller and Buchanan, but we do not believe this to be contradictory to their findings

Our equipment consisted of a platten with a small hole (about 3/16 in dia) in the center A plunger, passing thru the platten, is balanced by adding water to a bucket at the opposite end of a horizontal lever The plunger is placed flush with the surface of the platten and its vertical position located by the cross hairs in a telescope The portion of tread where intensity of pressure was desired was then moved on the platten until it came directly over the plunger and load applied Weight is then added to the bucket until the pressure is balanced

Our work indicated that the intensity of pressure on a 5 00-19 tire with 800 lb load and 30 lb inflation pressure varies from 2½ to 3 times the inflation pressure This is considering a conventional 3 rib tire with design on the shoulders outside of the ribs—the pressure per square inch on the center rib varying from 2 3 to 2 6 times the inflation pres-

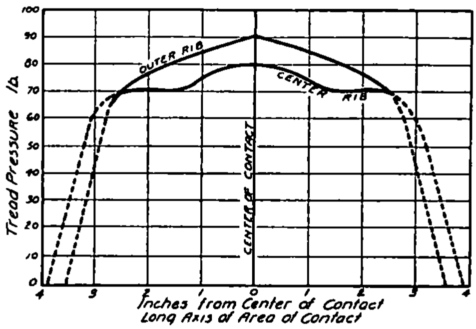


Figure 1

sure and the outer rib from 2 3 to 3 0 times the inflation pressure—varying from the center to end of contact area (See Fig 1.)

While the above work was on a passenger tire, it is assumed that larger tires would follow the same trend

Whether or not distribution over the area of contact would lead to significant differences in pavement stress is rather questionable However, this is more of a question for road engineers to decide

THOUGHTS ON HIGHWAY DESIGN RESEARCH AS RELATED TO SAFETY OF VEHICLE OPERATION

By Charles M Noble

Assistant Engineer, The Port of New York Authority

SYNOPSIS

Possibilities of research are outlined for the purpose of establishing definitely the basic causes of motor accidents and the relation of highway design to accidents The point is stressed that if those basic underlying causes can be known which originate in some subtle defect in the design of the highway and in the psychology of the driver, engineers can surely devise effective means to reduce such accidents as are chargeable to the highway itself, and at the same time correct and prevent many errors of the motorist Before highway designers can design for basic safety, enough of driver psychology must become known to enable the designer to arrange conditions in such a manner that the driver will instinctively choose the safe act The striking results of research, education and enforcement in the reduction of industrial accidents is cited and the analogy existing between that effort and the present motor accident situation is noted

In the field of technical design of those highway elements effecting safety of operation, the uncertainty of design features is treated and suggestions made relative to the desirability of research for the purpose of definitely establishing design practice The features mentioned include the width and detailed treatment of the center safety dividing space of dual highways, the lack of knowledge on permissible safe speeds for particular designs, lack of knowledge with respect to shoulder widths, spiral curves and distance between reverse curves, and the effect of vertical curves on headlamp visibility and entrance and exit facilities

The effect of the time element on design of modern highways because of the advancing speed of motor cars is outlined, elementary examples are given illustrating the thought that design features should be located at distances from the driver expressed in seconds of time, and necessary research is suggested to establish the data needed for designing with confidence

With some notable exceptions the bulk of past highway research has been in the field of the physical properties of the materials of construction, and little attention has been directed toward ascertaining the basic requirements of highway design as related to the safe and efficient operation of motor vehicles, with the result that great progress has been made in physical design and construction while insufficient knowledge is available in the realm of design affecting safe operation Thus, a challenge is offered the research worker, and fertile ground is available to organizations with sufficient financial and technical means to cultivate this form of investigation

The appalling accident and death rates indicate that many problems in highway design are unsolved The proper solution of these problems is vital, for the ease and comfort with which a modern motor car can be operated at high speed has revolutionized the requirements of design to such an extent that a national emergency exists One need only mention that more persons are killed in motor accidents in one month than have perished in all the floods in the United States within the past ten years, to indicate the magnitude and seriousness of the situation

Before it will be possible to establish confidently the fundamental principles of safe highway design, it will be necessary to ascertain the basic causes of accidents, to establish the relation and effect of highway design on accidents and to uncover the mysteries of driver psychology The conventional method of studying police accident reports, reveals only the most obvious accident causes, and frequently a wholly misleading picture may be obtained, for often the real and fundamental causes are so obscure and subtle that they will require the talents of a particularly trained and endowed investigator to uncover[1] All too frequently blame is attached to one or both drivers with no attempt being made to ascertain whether highway conditions created the circumstances which caused the accident This attitude on the part of the police is understandable when it is remembered that it is more natural to blame an individual than an inanimate facility, particularly since the participants may appear in a court of law to recover damages, and, since it is not possible to recover damages from an inanimate highway, the emphasis has been to search out the human error involved rather than to look back of the human error and find out what caused that error, for many times a particular set of inanimate components of the highway combine to cause the motorist to commit an error It is the search for such fundamental causes that concerns the investigator and from which will emerge a clearer conception of the relation of highway design to accidents

It may be of assistance in grasping the writer's meaning, to realize that almost never is an accident deliberate Seldom does a driver deliberately set out to cause an accident In other words a motor accident is truly an accident This suggests that it is very possible for highway or street conditions to be the fundamental originating cause of many accidents, conditions which at present are not recognized as hazards, things so subtle that they are not even suspected Unraveling this mystery will require a high type of detective work The situation is somewhat analogous to the condition existing in industry a number of years ago when the industrial accident rate was quite high At that time accidents were assumed to be an unavoidable evil and part of the risks of manufacturing, little effort being made to improve the situation Then, as today in the motor field, the operator of the machine was assumed to be at fault in case of injury, the assumption being that he was inept or careless Fortunately, men of vision believed that many accidents could be avoided by a study of causes, by education and by regulations forcing operators to conform to safe practices, and these methods have been so successful that all industrial accidents have declined materially, in spite of increased production, while in certain industries the accident rate has fallen to miraculously low levels Thousands of men and women are alive and well today who would be dead or permanently disabled had it not been for that effort This is a remarkable tribute to the men and organizations responsible for these results, particularly when it is remembered that in the beginning the very persons benefited most vitally, the workers, were almost entirely indifferent, and in many cases refused to cooperate

A similar opportunity exists in the highway field, particularly as related to causation and to the psychology of the driver If it is possible to ascertain the basic originating cause of an accident and establish the fundamental concepts of driver psychology, then, and only then will it be possible to design the highway for basic operating safety insofar as physical and psychological conditions influence accidents Many of you are no doubt familiar with the psychological investigations conducted a number of years ago at Ohio State University[2] Those studies were made in an effort to dis-

[1] Tentative qualifications of the proper type of investigator (1) Practical experience in highway design and construction, (2) Thorough education in the physical sciences, (3) Extensive training in advanced mathematics, (4) Education in psychology, (5) The mental perception to realize that the above are only tools and are a means to an end, not the end itself, and, (6) Practical common sense

[2] Weiss and Laurer, "Psychological Principles of Automotive Driving," Ohio State University 1927-29

cover the suitability of the driver for the car and the highway Furthermore the manufacturers of motor cars also regularly subject the vehicle to exhaustive tests to determine its suitability for the driver and the highway

Tests are as urgently needed to determine the suitability of the road for the driver and the vehicle, for engineers will not be able to perfect the technique of safe highway design until all the basic facts of driver psychology and the physical effects of road conditions on accidents are known Before highway designers can design for basic safety, enough of driver psychology must become known to enable the designer to arrange sets of conditions in such a manner that the vehicle operator will unconsciously choose the safe act rather than react in a way to lead up to an accident

There are many who will contend that such results are impossible, but, since that particular state of mind has been proven incorrect in connection with almost every major accomplishment in the scientific and engineering fields during the past 100 years, the public has come to expect the impossible from science as a matter of course To illustrate, not long ago foundation engineering was in a condition somewhat similar to the highway problem of today Foundations were designed by rule of thumb, based on personal experience, and a scientific approach was considered unattainable until Dr Terzaghi appeared and laid the foundations of soil mechanics The complexities of the problem were so great, conditions so varied and intricate that the idea defied the imagination, and many practical engineers thought a solution impractical or impossible But Terzaghi persisted, with the result that his principles have been enlarged and extended by other able investigators, practical design is utilizing soil mechanics and today he is acclaimed throughout the word as the father of a new science

The accident problem is in urgent need of a similar type of fundamental analysis and research, for if engineers can but know what is causing accidents there can be little doubt that engineering ingenuity can surely devise means to combat them, with the result that thousands of lives will be saved and the tremendous economic loss occasioned by motor accidents will be materially reduced The motor accident problem is more difficult and serious than any that engineers have ever been called upon to solve Research into physical materials is simple in comparison, because engineering materials follow definite and constant laws, however obstruse, intricate and difficult of perception, but the accident problem involves human behavior and reaction in almost infinite variety and complexity Yet there must be certain patterns, certain similarities and certain "laws" which will govern in many instances and of which use can be made in the design of highway facilities It is, therefore, earnestly hoped that the opportunity will be seized and that searching and basic research will be undertaken and vigorously pursued until the fundamental principles of accident causation are definitely established

Unfortunately the other tools effectively used by industry to reduce accidents, namely education and regulation, have not been as effective in the motor field as might be hoped (except in fleets of trucks and busses), for whereas industry has a certain amount of control over its employees and can force them to adopt safe practices, and may in addition weed out the accident prone, such a course has not yet proved practicable in the motor accident field This may be attributed to the temperament of the American people, which is known to be impatient of regulation in any form, and since the public is not an employee the present means of education and enforcement have not been conspicuously successful

Therefore, since this form of accident prevention is equally as important as prevention by safe highway design, it is as fundamental that research be instituted for the purpose of discovering means whereby the public can be persuaded or forced to adopt safe driving practices This thought is not new to business, for sales and advertising groups have become expert in persuading the public to purchase their products and they have developed a reasonably efficient technic which is based on research as well as experience The identical methods used by business to persuade the public to buy their products may not be effective in the motor accident field but undoubtedly research will develop far more efficient methods than now employed for this purpose In addition, research for developing positive means of enforcement is urgently needed, for there is always a small group of individuals on whom the most efficient and scientific methods of persuasion and education would be wasted

Fortunately an attack has already been started at Harvard University on this phase of accident prevention, which shows great promise of success, and it is hoped that this work will be further encouraged and stimulated

If, then, research can uncover the fundamental causes of accidents and indicate the relation between highway design and accidents, while at the same time effective means of education, persuasion and enforcement of safe driving practices are developed, a great stride will have been made toward making motoring on American streets and highways safe

In addition to the subtle and psychological a number of unsolved problems relating to design are awaiting much needed research in the physical field For example, can anyone state with certainty the correct width and treatment of the safety dividing space for a dual highway to satisfy particular sets of conditions, and can anyone specify with certainty the maximum safe speed that vehicles can operate on any given type of design?

From a practical standpoint it must be admitted that financial and physical circumstances will limit the width of the space dividing dual highways On the other hand the average motorist, if questioned, would undoubtedly indicate a preference for a design without curbs, flanked with wide smooth stabilized shoulders Such a design is practical and safe, when a wide separating space is used, but as the width decreases it becomes evident to thinking engineers, that, at a certain limit, some type of design is required which will prevent impatient drivers from using the safety space as a means for passing (possibly simultaneously in opposite directions, resulting in head-on collisions), and as a means of a certain amount of protection to opposite direction traffic from cars out of control Studies are urgently needed to determine the limiting width when protective devices are required in the center, and to indicate the type of detailed design most suitable for certain widths For example, on an elevated structure where every increase in the width of the structure represents an appreciable increment to the capital outlay, the designer is faced with the choice of making the width as small as possible, which may take the form of a low wall one (1) foot wide, or the provision of a greater space of some arbitrary width If a greater width is used, the problem resolves itself into a choice of normal height vertical curbs, high vertical curbs and low sloping curbs, all types with or without a low wall in the middle At present there are no data on which to base a detailed design or to determine the minimum safe width

The type of curb is also a problem where conditions are such that the width need not be an absolute minimum It is recognized that a vertical curb over 3 or 4 in high will prevent motorists, pinched by traffic, from mounting the curb and thus it may be expected that sideswipe accidents will increase where high vertical curbs are used The motorist senses such hazards and instinctively drives clear of high vertical curbs, usually a distance of from 4 to 6 ft, thus causing an economic waste in the effective width of the highway If the designer wishes to use low or sloping curbs he has no certain knowledge of exactly what type to use for it is quite possible that he may use a type which might throw a vehicle completely out of control if mounted at even a moderate speed, or that vehicles may be unable to mount them when covered with ice Again, the type used may offer so little obstruction that drivers will at the least provocation mount the curb and use the space for passing, thus greatly increasing the hazards of operation

In determining the answers to all the above detailed problems, opinion is not of much value A canvass of one hundred engineers would, in all probability, result in one hundred different solutions Therefore, the only sure basis of determining the true facts is from a thoroughgoing investigation in which the accidents occurring on each type of design are analyzed and exhaustive practical tests are conducted simulating actual accidents and noting the resulting damage created by each type of design In this way, the ideal design which subjects passengers and cars to the least injury would soon be indicated

The question of the safe permissible speed which may be used with a particular design and width of separation is an entirely unexplored field and thus it will not be possible for engineers to design for over all economy or to post safe speeds until research provides the answer to this question Therefore, it is most urgent that such investigations be instituted before any considerable program of dual highway construction is undertaken

The safe speed that a vehicle may travel is influenced to a considerable extent by the width and condition of the shoulder adjacent to the pavement, but as yet there is no definite method of evaluating in concrete terms its effect on motor safety In other words, because of lack of knowledge, the value of the shoulder is an unknown intangible, and it is generally conceded that engineers dislike dealing with intangibles It is therefore evident that more knowledge is required to establish the facts relative to the effect of the shoulder on safety and on safe speeds To illustrate, if the highway had no shoulder, with an abrupt drop at the pavement edge, the average motorist would feel quite unsafe and it would be readily admitted that the safe speed should be quite low, cars stopping for any purpose, such as changing a tire, must of necessity remain on the pavement, thus completely blocking the lane, and all drivers must observe extreme care not to run off the edge of the pavement, for such a course would immediately result in a serious accident Suppose an increment of one foot is added, making a shoulder one foot wide, the question awaiting an answer is, how much has this increment added to vehicle safety and what may be the safe speed? It is probable that as increment after increment is added to the shoulder width, the locus of a curve on a diagram representing benefits would continue to ascend until a certain width has been reached after which the curve will droop over and start to descend, indicating that after reaching a certain width any further increase of the shoulder follows the law of diminishing returns

To the writer's knowledge no start has been made on this problem, and yet it is quite as vital as many other highway problems which have been exhaustively investigated It must be admitted that the problem is not simple but highly complex, yet, intelligent, careful and patient research is certain in the end to provide the engineer with tangible facts on which to base a design for the various types and classifications of highways he is called upon to construct

Considerable discussion has been directed to the problem of setting up a proper criterion of the length of spiral curves A recent method of approach has been on the basis of the rate of change of centrifugal acceleration, but there is some question as to whether this method will really lead to a proper solution for it is quite possible that the rate of superelevation and the friction value assumed in the formula for superelevation may have as important an influence on the spiral length as the rate of change of centrifugal acceleration A statement of the case may make the issues clearer

In order for the motorist to feel the effect of centrifugal force and the rate of change of centrifugal acceleration, the curve must not be superelevated, or the vehicle must be travelling at a higher rate than the speed for which the curve is banked In other words, if a vehicle is travelling around a curve at the exact speed for which it is banked, the operator will not be affected by centrifugal force since the force of gravity introduced by the tilt in the roadway cancels centrifugal force, and if a spiral is utilized which at every point is banked for the given speed and radius at that point there will still be a balancing of forces even if the spiral is only one foot long However, a rotational force is introduced by the transition from the ordinary crown to the superelevated section and if the spiral were only one foot long this rotation would be too rapid for comfort or safety In the case where the vehicle is travelling at a faster rate than the speed for which the curve is superelevated, then the forces introduced will amount only to the difference in speed between the superelevated speed and the actual speed, and since the speed is cubed in arriving at the rate of change of centrifugal acceleration, the difference in the forces is appreciable A numerical case will illustrate

$$L_s = 1\,58\,\frac{V^3}{r},$$

in which L_s = length of spiral, v = velocity in M P H , and r = radius in feet

(The formula is based on a rate of change of centrifugal acceleration of 2 ft per sec per sec per sec , and assuming no superelevation) If it is assumed that the rate of banking E is 05 ft per foot of width, permissible friction F 03 and speed 60 M P H then from the formula $E+F=\frac{067V^2}{r}$, r = 3000 ft (in round numbers) Substituting these values of speed and radius in the spiral formula, $L_s = 113$ ft If, however, only the difference in the actual speed and the superelevation speed, which amounts to 13 M P H , is introduced into the spiral formula, $L_s = 1$ ft Thus, it is evident that if the actual unbalanced force is used, an entirely inadequate spiral is indicated, while,on the other hand if the full force is used (balanced plus unbalanced) there is serious question if consistent riding qualities will be produced with different friction values and different rates of superelevation

A method suggested by Elmer R Haile, Jr ,[3] which utilizes the rate of rotation as a measure of spiral length may yield more consistent results This method requires the substitution of two values to be derived from tests

In view of the uncertainties outlined it appears desirable that experimental spirals be constructed, embodying the different conditions encountered in practice, and that they be subjected to scientifically controlled tests to the end that

[3] *Proceedings,* American Society of Civil Engineers, December 1936, p 1650

a real and satisfying criterion of spiral length may be set up It is also important that it be determined if very long spirals are detrimental, or are desirable

To the mathematically inclined, there is opportunity for service in the invention of a mathematically simple spiral for special use on elevated viaducts and in tunnel construction, in order that concentric curves and radial lines may be accurately determined for the purpose of shop fabrication of steel

In the past, the minimum distance between reversed curves has been established more or less arbitrarily, but with advancing speeds it becomes necessary to replace past methods, based on judgment alone, with definitely known facts founded on scientifically conducted tests The minimum distance between reversed curves depends on the reaction time required for the operator to emerge properly from one curve and become prepared to enter the following one The reaction time in seconds multiplied by the vehicle velocity in feet per second at the design speed will give the minimum distance in feet between the curves It is quite possible that the friction assumed as permissible in the superelevation of particular curves will have an important bearing on the reaction time and that the use of spirals will still further affect the time, due to the fact that steering (or slippage) angles are released gradually on passing out of the first curve and developed gradually on entering the reversal

Since the basic facts concerning the reaction time required between reversed curves for the various design assumptions of friction and spiral length are unknown, it would seem fundamental that authoritative data be accumulated based on tests in order that this detail of design may pass from the field of uncertainty into the realm of definite knowledge

The lengths of vertical curves affect visibility and sight distance This has generally been recognized and taken into consideration in design in relation to sight distance during daylight, but little thought has been given to the problem as related to sight distance with headlamps during the hours of darkness Indeed, at the present time, it is not possible to set up the minimum length of a vertical curve for a given algebraic change in grade which will assure full visibility with headlamps while at the same time not result in a curve of a greater length than actually required to fulfill practical requirements A mathematical solution may quite readily be developed, provided certain facts relating to lamp characteristics are known, as for example, the height of the lamp above the road, the vertical angle of the upper rays of the beam and the length of visibility on a level road, but the value of a mathematical approach is questioned because of the extreme length of curves that are indicated

Until recently, the length of visibility with a headlamp was unknown but fortunately certain facts have been made available by W C Giessler[4] as a result of a series of tests conducted by the Illinois Division of Highways The striking fact was disclosed that it is possible with properly adjusted headlamps, in good condition, to see a white object at a distance as great as 715 ft from the car

The vertical curve problem is divided into two parts (1) At sags, and, (2) at summits If it is desired to utilize vertical curves which will not restrict headlamp visibility for a distance of 715 ft, assuming the height of the lamp to be 3 ft, the upper aiming to conform to legal requirements (Pennsylvania, 2° 40′) and

[4] "The Relationship of Headlamps to Road Speeds"—by W C Giessler, Mechanical Engineer, Illinois Division of Highways (February 1937)

"What is the Night Accident Problem Like"—by W C Giessler (May 1937) the algebraic difference in the highway profile to be 8 per cent, then a mathematical solution indicates that at a sag a vertical curve 560 ft long is necessary, while at a summit a vertical curve 6800 ft long is required The length of the latter curve is so great that serious doubt exists as to whether curves of such length are really required to assure full headlamp visibility A further analysis of both cases may indicate the general features of the problem

used instead Mr Giessler found that maximum visibility was obtained with the top of the "hot spot" located 2 in above the horizontal axis of the lamp at a distance of 25 ft, which represents a vertical angle of 0° 23′ The result is that a longer vertical curve than indicated above is required to maintain the sight distance available with headlamps, the calculated length being 2640 ft

Figure 2 shows the conditions at a summit It is to be noted that the round-

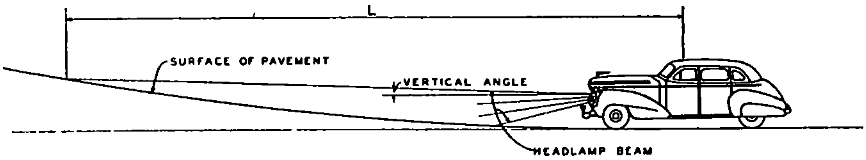


Figure 1 Condition at Sags

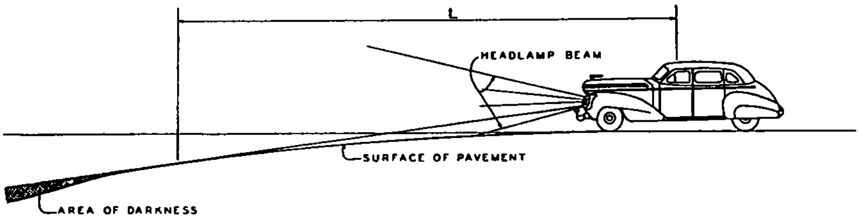


Figure 2 Condition at Summits

Figure 1 shows the conditions at a sag The mathematical solution mentioned assumes that the light is projected forward with equal intensity and that the top rays will reveal objects at as great a distance as the center or "hot spot" rays This does not conform to common observation, as it is known that the edges of the light rays become shadowy and that objects are not defined so clearly in the outer areas as those within the center of the intense rays at a corresponding distance from the car Thus, it can readily be seen that the vertical angle of the extreme upper rays should not be utilized, but that the vertical angle of the intense or "hot spot" rays should be

ing curve intercepts the light rays and that the down curving pavement beyond the tangent point is in darkness In other words, there is a sliver of darkness below the bottom of light rays and the top of the pavement beyond the tangent point In the example given above it was assumed that the tangent point should be at the distance of maximum headlamp visibility from the car, namely 715 ft With a curve of this length, the rate of change or "rounding" of the curve is quite slight and thus the sliver of darkness beyond the tangent point is very thin, in fact at a distance of 100 ft beyond the tangent point the depth of the darkness is only three quarters of an inch In view of the fact that the "hot spot" must be aimed above the horizontal for maximum visibility, and that the lower fringes of the light are of lower efficiency, it is quite possible that the lower fringes of the light will be shadowy on level pavement at the maximum distance and thus the thin sliver of darkness for a curve of such great length may cease to govern visibility requirements after the vertical curve is increased beyond a certain magnitude This suggests that shorter vertical curves than indicated by a mathematical solution may possibly be used at summits without adversely affecting sight distance with headlamps

In view of the almost complete lack of definite knowledge on the subject, it appears only reasonable to assume that design standards must await the results of tests and it is to be hoped that such an investigation may be instituted in the near future

An important detail which is yet in the development stage is the design of entrance and exit ramps to and from high speed highways, particularly in urban regions where space is restricted It is important that these facilities function efficiently, while at the same time, assuring the maximum safety not only to the entering or leaving vehicle but particularly to the vehicles on the high speed highway where an accident is of a more serious nature because of the speed involved The fundamental principle of acceleration and deceleration lanes is being adopted in modern designs, but their length for particular situations and design speeds and the details of design are in the experimental stage It is to be hoped that studies will be instituted for the purpose of establishing basic principles and to indicate definitely bad or undesirable design Such principles cannot be established from opinion, but must be founded on facts deduced from accident analyses, congestion studies and habits of drivers obtained by investigating the various types of design in actual use today. In this connection, it would be helpful in determining natural travel paths if the natural radius which an operator utilizes of his own volition when making a turnout for passing on the open highway is established; and the rate of deceleration of passenger cars and trucks in gear with the throttle closed utilizing the compression in the engine as a braking force It must be realized that a traffic stream is similar to a flowing liquid, and abruptly introducing new streams of flow causes turbulence. An entrance or exit ramp causes such turbulence unless it is designed correctly An entrance ramp, for instance, suddenly reduces the number of main travel lanes each time a vehicle enters unless the vehicle is given an extra lane of its own for such a distance that it can accelerate to the travel speed of the other cars and mesh into traffic without turbulence In other words, the efficiency of the main highway is seriously impaired unless proper exits and entrances are provided, and thus it can be seen that improper design of these facilities adversely affect the functioning of the entire highway, and, therefore, they are of vital importance in maintaining balanced operation of the complete system as well as affecting the safety of the motorist

The element of time is the new factor which has become increasingly important due to the advancing speed of the modern motor car Therefore, designers must begin to think in terms of highway elements being arranged or placed so many seconds away from the driver That is to say, if the time required for the driver to react to a given condition is known, then the required distance in feet for locating a design feature is obtained by multiplying the reaction time in seconds by the design speed in feet per second The difficulty at the moment, however, is that very little data are available in print on the reaction time required for many motoring situations, for although machines for measuring reaction time for various situations have been developed, it must be recognized that the subject operating such a machine is keyed up and alert, thus being able to perform a given function in a shorter period of time than is usual under actual operating conditions In other words, the motorist under actual driving conditions is seldom ready to go into instantaneous action when an emergency arises and therefore requires a longer time to react than when being tested by a time measuring machine This suggests that vital information is still required before engineers can confidently design to conform to the operating characteristics of motorists The striking similarity of many serious accidents is the lightning like rapidity with which they occur Many participants are unable afterwards to give a clear picture of the events which led up to the accident and often have not the faintest idea of what really happened This is because the action took place in a split second and was completed before the driver's mental machinery could grasp what was happening This imposes on the highway designer the task of lengthening out the perspective ahead of the driver and of arranging the highway elements so that time and space are provided in such a manner that it is possible for the motorist to correct an error in judgment or to recover from a minor mishap in time to avert catastrophe A statement of a few of the more simple and obvious cases in which the time element affects design may be illuminating

A not infrequent accident condition is the case where a passing vehicle cuts in ahead too quickly and contacts the front of the vehicle being passed sufficiently to cause momentary loss of steering, and if the pavement is flanked by a narrow 5 foot shoulder with ditches, trees, poles, headwalls, etc, at the shoulder edge, the driver must be able to recover control within approximately $\frac{1}{4}$ of a second (Speed 40 M P. H) in order to avert a major accident In this case the shoulder may be said to be $\frac{1}{4}$ of a second wide At the present moment it is not known what reaction time is required for a driver to recover control of steering under such circumstances or the time consumed by a car out of control in traversing lateral space

The design, size of lettering and position of a direction sign depend on time

Frequently a motorist passes a direction sign before being able to read it, or if the letters of the sign are of sufficient size to be read it may be placed improperly in such a position that the motorist has passed the turnout before he has had time to react to make the turn For example, if the speed is assumed as 60 M P H and the reading and understanding time 3 sec, the letters of the sign must be of such a size that they can be easily read at a distance of 264 ft In this case the sign may be said to be 3 sec from the motorist The sign also should be set sufficiently in advance of the turnout for the motorist to make up his mind if he wishes to make the turn, for brake reaction time and for decelerating at a moderate rate to the speed required for the turn If those functions require 5 sec (440 ft at 60 M P H) the turnout may be said to be 5 sec from the sign The fact that many direction signs have letters of improper design and size to be read and understood at as moderate a speed as 40 M P H, and that they are placed an improper distance in advance of the turnout is an indication that the factors are either unknown or that they have not been widely disseminated

Hazards such as wells separating turnout ramps from the main roadway should be properly located and the motorist protected from them by allowing adequate warning of their existence and then an allowance of sufficient time after the warning to enable the motorist time to avoid crashing into them This protection and warning may take the form of an island extending out in front of the wall for some distance, the end of the wall illumined with an indirect outdoor "billboard" type of light, the end of the island to be low so that it in turn may not be a hazard and with no form of fixed obstruction located in the island such as light standards, reflectors and signs The length of the island from the wall to the tip should be such that a motorist who straddles the island will have time either to come to a stop or swerve off before striking the wall If the island is low enough to permit swerving, the time required will be the sum of two reaction times, first a period to come to the realization that danger is ahead and that there is necessity for action, and second, a period sufficient to swerve out of danger If 3 sec are required (176 ft at 40 M P H) then the tip, or nose, of the island may be said to be 3 sec from the wall In the case where the sides of the island are designed with curbs of such height that it is not possible for a vehicle straddling the island to swerve off, the time required will be the sum of the reaction time for the motorist to come to the realization of the danger and of the necessity for action, the reaction time for brake application and the time required to bring the car to a stop before reaching the well If 7 sec are required (approximately 275 ft at 40 M P H), the tip of the island may be said to be 7 sec from the wall

The reaction times required for these conditions are unknown and in addition they may be different from similar reaction times occurring under favorable weather conditions, for the hazards imposed by walls and other obstructions operate during periods of darkness, fog, rain, sleet and snow when visibility is poor

It may be thought that many questions raised in this paper are only a matter of detail and are consequently subordinate to other features of highway design, but it must be realized that other elements of the highway, particularly those features involving strength of construction, have been receiving considerable attention over a period of years, while knowledge of the questions treated herein is quite vague and unsettled, yet such problems have a vital effect on motor safety Consequently the motorist will be appreciative of their solution since he is quick to sense any advancement in highway design which improves vehicle operation and safety

It is well to note that it has been stated in structural circles that close attention to details is the secret of success in structural design This thought is even more true in highway design, but the fact has only recently been recognized, particularly in relation to safety of operation Perfected highway design awaits definite and precise data concerning these details, and it is earnestly hoped that research will be undertaken to clear away the uncertainties in order that the engineer may proceed with certainty to design safe and efficient highways The results will be measured in lives saved and in a tremendous reduction of the economic loss due to accidents

DISCUSSION ON DESIGN RESEARCH

MR BURTON MARSH *American Automobile Association.* Mr Noble's paper deals with a feature of highway design which is bound to get a great deal of attention—the matter of the divided highway It was pointed out that much needs to be done about the design of the center strip, its curb, planting, etc These design features do need careful study

I had occasion about five years ago, in connection with a paper on traffic features of highway design, to interrogate highway engineers concerning their attitude toward divided highways, which I favored I found relatively little support for this design Recently in preparation for another paper on a somewhat similar subject, a similar inquiry was sent to the state highways departments Responses indicated an almost unanimous favor for this type of improvement In fact, there was only one negative response among the 42 replies received In light of this attitude of approval and despite the fact that at present there are now but 1,200 miles of non-urban divided highway in the country, it would seem that here is a type of improvement for highways four lanes in width or wider, which will be increasingly utilized

It is, therefore, particularly appropriate that such important questions which relate to highway use should be given attention by progressive engineers Demand for divided highways is certainly increasing and efficient designs should be developed Highway users will profit greatly if active discussion and research precede extensive construction involving this feature.

DR A R LAUER, *Iowa State College* Mr Noble mentioned reaction time Perhaps we are wrong in allowing for an average of one-quarter, one-half or even three-quarters of a second I believe we should think of such matters in terms of the maximum time that anyone is likely to take, then add still more as a margin of safety In designing a bridge we allow for the maximum load that will pass over the bridge, and a wide margin of safety in addition

I feel this is an important point in the matter of sight distance We should consider only the maximum time any individual is likely to take in perhaps but 1 out of 20 instances It is the exceptional individual at the inopportune time which causes all the trouble.

To attempt to build shoulders wide enough to protect the slow-reacting driver, when he finds himself headed for a ditch, would be utterly impossible You would need shoulders wider than the main road as some drivers may even "freeze" under conditions of fright and take 1, 2 or even 3 or more sec to regain control of themselves This is exceptional Perhaps one sec basic time would be sufficient in most cases This might be doubled to include an adequate safety factor

REPORT OF THE JOINT COMMITTEE ON ROADSIDE DEVELOPMENT OF THE AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS AND THE HIGHWAY RESEARCH BOARD

By H J Neale, *Chairman*
Landscape Engineer, Virginia Department of Highways

(In Abstract *)

On October 1, 1937, eleven States were actively engaged in cooperating with the Soil Conservation Service of the United States Department of Agriculture on 71 erosion control cooperative research projects, and others have indicated progress in this direction Several States, including some outside of the Soil Conservation Service regions, have undertaken soil erosion projects in accordance with the outline prepared by the Committee The Soil Conservation Service, in recognition of the importance of this work, has assigned one of its technicians to coordinate highway erosion control research projects throughout the country

According to the best records available, out of approximately 150,000 miles of improved highways on the federal aid system only about 5,000 miles have received intensive roadside development Relief labor could be utilized effectively on this vast remaining mileage as it has been demonstrated that such work provides for extensive maximum use of labor without excessive expenditures for materials and equipment

* The complete report of the Joint Committee for 1937, including papers presented at the Seventeenth Annual Meeting of the Highway Research Board has been published in a special bulletin, "Roadside Development," March 1938

In order to meet more adequately the constantly growing demands of over 24,000,000 passenger car owners for safety and enjoyment of the highways, more consideration should be given to landscape advantages, together with the acquisition, design and development of wayside areas at strategic locations

Consideration should be given not only to the protection of the highway investment, now running into billions, but also the conservation of enormous land values adjacent to these highways

To effect savings in maintenance operation, there should be more integration of approved roadside development practices in design and construction

The Committee hopes that means may be found for the establishment of research fellowships in certain strategically situated universities or agricultural experiment stations, for the purpose of studying special interrelated problems of soil conservation and erosion as related to highway practice

The Committee acknowledges the cooperation being furnished by the U S Bureau of Public Roads, Soil Conservation Service, Tennessee Valley Authority, National Park Service, and the various highway departments in its research studies and in the sponsoring of uniform roadside procedure

THE DESIGN OF THE HIGHWAY CROSS-SECTION

By Wilbur H Simonson

Senior Landscape Architect, U S Bureau of Public Roads

SYNOPSIS

Analysis has been made of from 1,200 to 1,300 individual cross-sections of primary 2-lane rural highways selected from 50,000 projects submitted by State highway departments during the 8-year period 1930-38 Based on averages for 1932, 1934, and 1936, with forecasts for 1938 and 1940, a composite picture is presented of current trends in the design of the highway cross-section for both low- and high-type roads From the steep slopes of 1932, with sharp angular intersections, there appeared in 1934 the first indications of the effect on highway construction brought about by the introduction of roadside improvement programs By 1936 flatter and rounder slopes, wider traffic lanes and shoulders, shallow gutters, sidewalks and hundred-foot right of ways had become increasingly evident In 1938 the probable trends for tomorrow's highways indicate that safety, utility and appearance will be enhanced by the application of landscape principles in highway cross-section development A balanced relationship may be expected among the three divisions of the highway cross-section the traffic area, the roadside (public) area, and the adjacent private lands New road undertakings will take careful account of present and probable future uses of each part of the highway cross-section so that the best appearing road will be the safest, and the safest road attractive in appearance

The design of the highway cross-section in relation to traffic safety is a subject that challenges the best efforts of all those interested in this problem Highway safety has become a public question of the first magnitude We know that the safety of traffic is influenced by a large number of factors, and that all accidents are the result of a combination of these factors, each contributing in some degree Adequate highway widths are recognized as one of the most important essentials for safety A review of cross-section practices now employed by the State highway departments to eliminate or reduce the dangerous highway conditions which are found to exist, and the gradual evolution of the standards of road design and construction which will provide the utmost of built-in safety should be helpful in determining the best design for the cross-section of tomorrow's highway

Previous studies of the highway cross-section have usually been limited to the data pertaining to the actual traveled way and have not been approached from the viewpoint of the right of way as a whole in relation to its surroundings Past efforts in this direction have aimed to cover certain portions of the cross-section only, such as the type, thickness, crown and width of surfacing or pavement, shoulders, guard rail, superelevation and widening, and similar engineering details of construction

The trend of highway design during the six-year period from 1920 to 1926, the close of the first decade of Federal-aid activities which began in 1916, was reported by A G Bruce, Highway Engineer, and R D Brown, Associate Highway Engineer, Bureau of Public Roads, in *Public Roads* for March, 1927 (Vol 8, No 1, pages 7 to 15) A résumé of the portions of that 1926 report which have a possible bearing upon the safety factors of highway development is given in the progress report of Subcommittee No 4 of the Joint Committee on Roadside Development covering highway types and roadside areas [1] Also, an article discussing the effect of increased speed

[1] The illustrations and diagrams accompanying this paper are based on the factual survey included in the Report of the Subcommittee.

of vehicles on the design of highways appeared in the March 1929 number of *Public Roads* (Vol 10, No 1, pages 11 to 20) These articles covering the trends in highway design of a decade ago furnish an interesting record for comparison with the present day trends in highway construction as covered by this paper

The changes which have been made in the design of highways during recent years appear as no less remarkable than the changes which occurred during the 1920 to 1928 period The general character of these more recent changes is perhaps already known to the majority of highway engineers, but it is thought that a review in some detail of the practices in use in each State highway department, and their evolution during the last several years will be of interest as an indication of current trends in highway design types This survey of facts will cover the 1930 to 1938 period, or the major portion of the second decade of Federal-aid activities in highway construction It may be of interest to note that the forecasts made in 1928 as to the roadbed features of highway design trends proved substantially accurate when checked with the 1930-1932 cross-section data presented below Except for one short paragraph indicating a noticeable trend at that time toward the acquisition of wider right of way, there was no indication in any of the earlier reports that roadside considerations had any appreciable influence upon the highway design of that period

A synopsis of the methods used in obtaining these source data will be found in the report of the Subcommittee No 4 of the Joint Committee on Roadside Development This will not be repeated here except to state that a representative "sampling" method was used and the data were limited to new construction on the primary two-lane rural highway, which type represents about 95 percent of the State highway mileage in this country Therefore, the conclusive tendencies indicated for this one major type should be far reaching in their application to future highway programs

The State highway systems comprise about 324,000 miles of highways that represent the principal routes of highway travel in the United States Figures given in the Annual Report of the American Association of State Highway Officials, giving the mileage of improved highways exceeding two lanes in width, as of July 1, 1937, show 4,704 miles of three-lane construction, 3,082 miles of four-lane construction and 221 miles of six-lane construction Of the 3,303 miles of four- and six-lane widths only 604 miles were indicated as of divided construction on which traffic in opposing directions was separated by a raised parkway or median strip

For convenience in presentation and comparison of progress, three representative construction periods were used, 1932, 1934, and 1936 The projected trends into 1938 represent the first fruits of the study to get a composite picture of current tendencies in the design of the highway cross-section The survey is based on the analysis of from 1,200 to 1,300 individual cross-sections selected out of a total of about 50,000 typical selections submitted by the several State highway departments during the eight-year period stated The facts here presented thus cover approximately a decade of relative progress in the development of the highway cross-section The gradual but steady growth toward a wider and wider cross-section is of particular significance now when transport is pressing for safer roads of greater capacity There is a distinct three-part pattern evolving in the typical cross-section of the future highway (1) The traffic area, (2) the roadside (public) area, and (3) the roadside (private land) area

The information showing past progress and present trends has been prepared in graphic form and it is believed these illustrations will be more or less self-explanatory without further discussion It will be noted that the trend in the construction of new surfaces built since 1930 on all roads and streets under State control, as taken from the annual compilations prepared by the Bureau of Public Roads, indicates that there was an average of nearly 29,000 miles annually of total new surfacing The trend in new low type surfacing represented an average *increase* of 1,000 miles annually, while the trend in new high type surfacing showed an average *decrease* of nearly 1,000 miles annually The graphic charts used in the presentation of the cross-section data show the two types separately in every case so that comparison may be made of both low type and high type construction "patterns" if desired

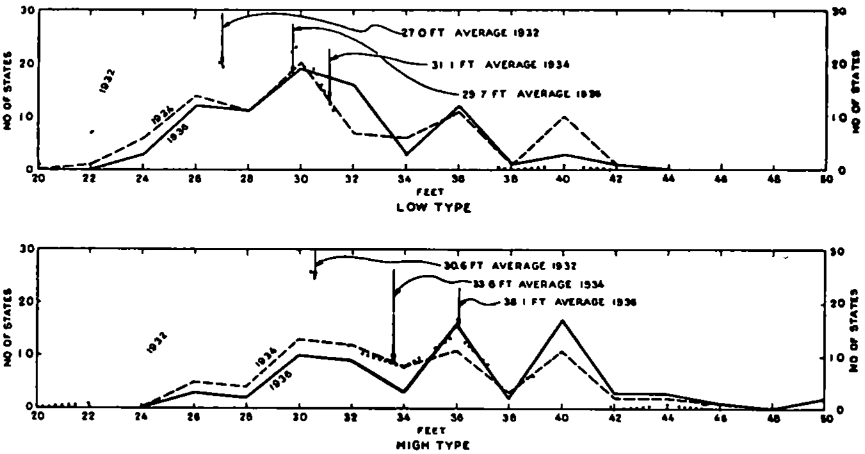


Figure 1 Trend in Widths of Graded Roadbed Used on Roads and Streets Under State Control

TREND IN WIDTHS OF GRADED ROADBED, FIGURE 1

Low Type The 27-ft average graded width in 1932 was increased approximately 3 ft to an average of 29 7 ft by 1936, when 30 ft was in greatest use with trends toward the wider 32 and 36 ft graded section evident in that year

High Type The 30 6-ft average for 1932 increased progressively through 1934 to an average graded width of 36 1 ft in 1936, with a dominant use of the 40-ft width indicated

TREND IN WIDTHS OF TRAFFIC LANES, FIGURE 2

Low Type The 9-ft lanes in greatest use in 1932 increased to 10 ft in 1934, to become a more dominant standard of construction in 1936, when a distinct trend toward the wider 11- and 12-ft lane width is evidenced

High Type The 10-ft lane width prevailed throughout the period reviewed, although the average of all lane widths used increased from 9 6 ft in 1932 to 10 4 ft in 1936 The slight use of 11- and 12-ft lane widths in four of the States in 1936 indicates a trend toward the greater use of these widths in the future

The average width of surfacing for two-lane rural highways of both low and high type increased approximately two feet from 18 5 ft in 1932 to 20 6 ft in 1936 The 22-ft surfacing to provide two 11-ft lanes is indicated as the future trend

Experience and observation both confirm the conclusion that a 10-ft traffic lane is no longer adequate for modern high-speed traffic For instance, the California Division of Highways recently adopted a new standard of construction for State highways which provides for an increased width of lane The present 10-ft lane is to be widened to a basic 11-ft width, making the two-lane roadway 22 ft wide instead of the previous 20 ft A standard of 11-ft width of lane for three-lane highways has also been adopted and these will be designed to provide for future expansion into four-lane divided highways with minimum loss of investment The multiple-lane highway of four lanes or more will be a divided highway providing for two roadways in each direction with a dividing or separating strip between them The standard of construction adopted for these roads is a 12-ft lane adjacent to the dividing strip and an 11-ft lane for the outside "The inside lane of 12-foot width," as quoted from the October 1937 issue of Florida Public Works, "will provide more freedom for the car traveling in this lane while passing and greater freedom and mobility in case of crowding The outside lane does not require this additional width since it has a shoulder still available to maneuver upon in case of necessity"

"Only a relatively small percentage of our highways will be of the divided type The majority of our roads will always continue to be two-lane roads since that width will accommodate the traffic requirements based on the character and volume of traffic using them Our planning of the narrower roads now constructed must consider the ultimate development or we will be forced to waste some parts of the pavement" (Developing Designs and Minimum Width for Strips are further discussed in September 1937 California Highways and Public Works, by C H Purcell, State Highway Engineer, in article entitled "Safer Highways")

A study of the passing of vehicles on highways as reported in *Public Roads,* Sept 1937 indicated that pavements of

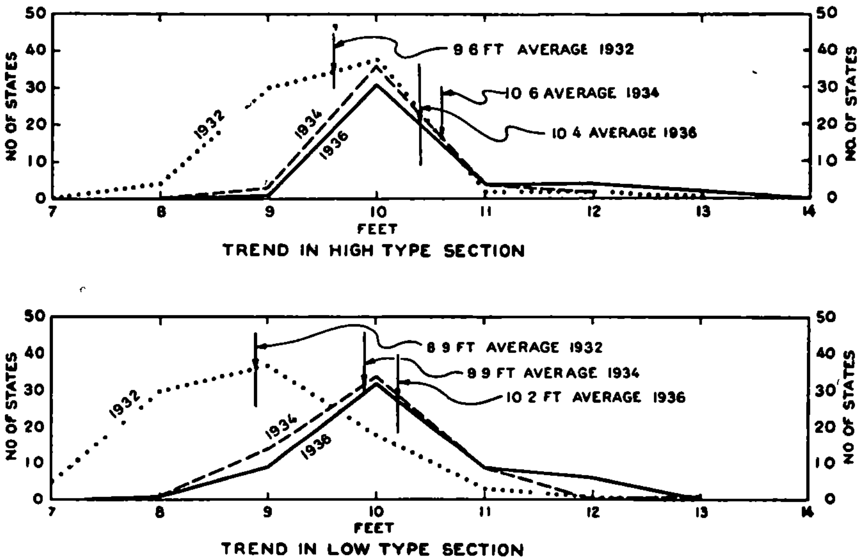


Figure 2 Trend in Widths of Traffic Lanes Used on Roads and Streets Under State Control

18-ft width are too narrow for modern passenger cars alone or for modern mixed traffic Pavements of 20-ft width are reasonably adequate for light-traffic roads used infrequently by wide trucks but are inadequate for heavy mixed traffic Pavements of 22-ft width are entirely adequate for modern mixed traffic However, as the speed of the critical vehicle increased, its distance from the right road edge increased This study concluded that, of any effects finishes are improved in the smooth uniformity of section, there is indicated a trend toward the 3/16 in per ft crown

High Type A crown of ¼ in per ft was in greatest use during 1932, but this was reduced to ⅛ in in 1934, and remained in favor throughout 1936 The indications are that this rate of crown will become more and more uniformly used The use of the flatter crown as a standard by a wider number of States is conducive to safety

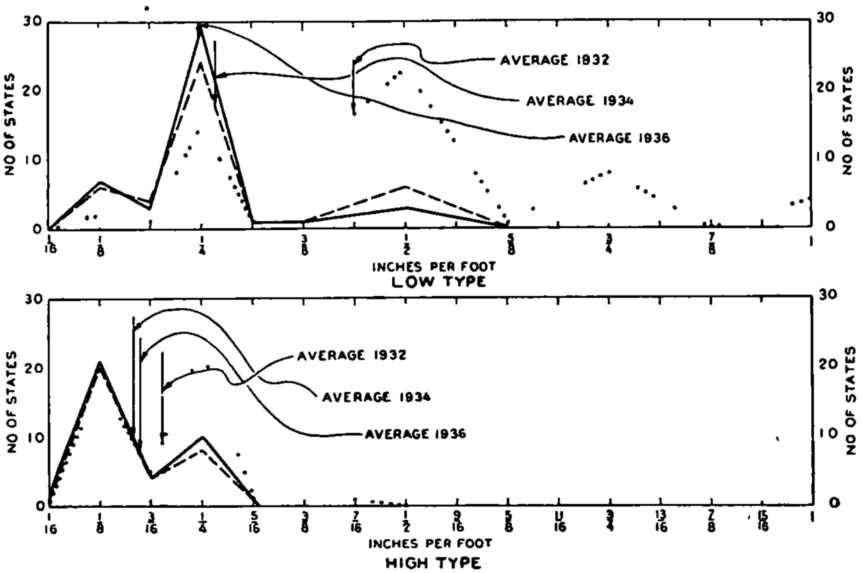


Figure 3. Trend in Crown of Surfacing Used on Roads and Streets Under State Control

speed may have upon vehicle position, the primary one is that involving greater edge distance Thus, further increase in the speeds of vehicles will tend to make additional road width necessary, which means 22 ft and sometimes possibly 24 ft of surfaced width

TRENDS IN CROWNS OF SURFACINGS, FIGURE 3

Low Type The ½-in per ft crown in greatest use in 1932 was reduced to ¼ in per ft in 1934 and 1936 As low-cost mixed in place and similar surface

TREND IN WIDTHS OF SHOULDERS, FIGURE 4

Low Type In 1932, the average shoulder was 4 3 ft wide This was increased (slightly more than a foot) to 5 6 ft in 1934, and remained at 5 4 ft average width in 1936 The 6- and 8-ft widths of shoulder are gaining in use

High Type The 5 9-ft average width of shoulder in 1932 increased nearly 2 ft to an average of 7 8 ft in 1936 While the 10-ft shoulder is now in dominant use in a large number of States, there is a trend toward even wider 11- and 12-ft.

shoulders The high-type shoulder seems to average about 2 ft greater width than used on the low-type construction slopes is in conformity with the tendency toward flatter crowns for highway surfacings, more closely uniting the surfacing

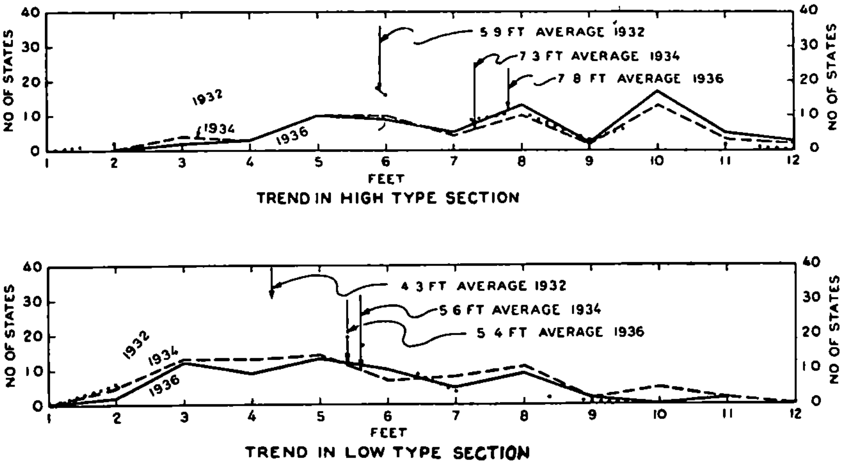


Figure 4 Trend in Widths of Shoulder Used on Roads and Streets Under State Control

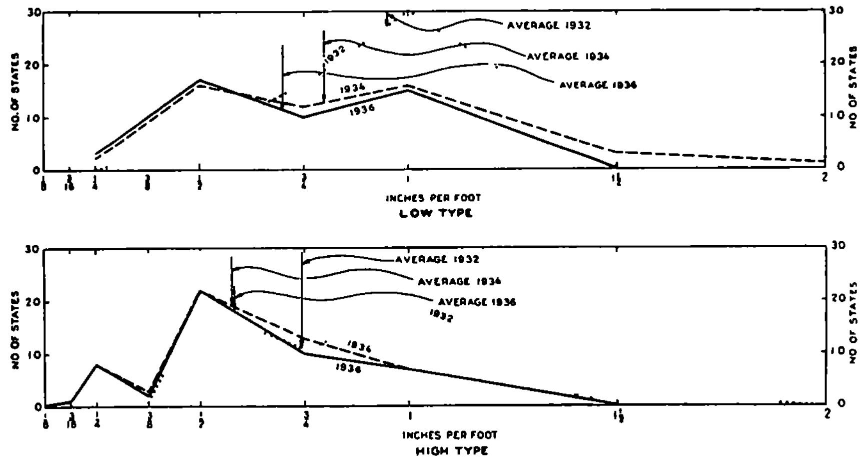


Figure 5 Trend in Slopes of Shoulders Used on Roads and Streets Under State Control

TREND IN RATES OF SLOPES OF SHOULDERS, FIGURE 5

All Types The present trend appears to be toward the more uniform use of ½ in per ft in slope of shoulder in both types, although ¾-in and some 1-in slopes are reported The trend toward the more general use of flatter shoulder with the shoulder However, while this reduction in the rate of slope may be favorable to lessened possibility of erosion, the tendency for grass or sod treated shoulders to grow above the edge of surfacing originally laid should be considered as it may interfere with free surface drainage from the pavement if the shoulder slope is too flat In such cases, the ¾-in pitch may be more suitable In any event, the rounding of the 2-ft portion of the shoulder width tends to merge the front slope with the shoulder as a unit for a more effective section

TREND IN TYPES OF SHOULDER SURFACINGS, FIGURE 6

High and Low Type Bare untreated shoulders appear to be most used in both

TREND IN WIDTHS OF GUTTER SLOPES, FIGURE 7

Low Type The 4-ft average width in 1932 had increased to an average of 6 7 ft in 1936, with 8- and 10-ft gutter slopes gaining favor

High Type The 1932 average of 4 ft increased to 6 6 ft in 1934, but this had fallen to an average of 5 7 ft in 1936 There is great similarity in the behavior of both types with regard to this unit

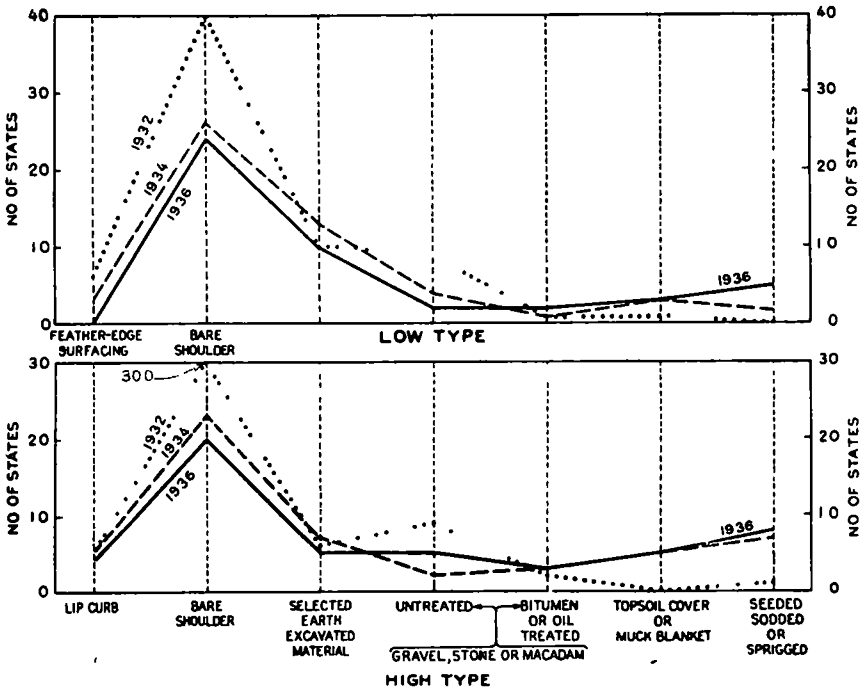


Figure 6 Trend in Types of Shoulder Surfacings Used on Roads and Streets Under State Control

types of construction There is now a trend toward topsoiling and seeding or sodding in a few of the States Since shoulder treatments such as oiling or graveling are largely performed by regular maintenance forces, the tendency in this direction might not appear in the construction cross-sections used as the basis for this survey

of the section While 6 ft is the dimension in greatest use, the trend toward 8- and 10-ft widths is definitely indicated

TREND IN PERCENT OR RATIOS OF SLOPES OF GUTTERS, FIGURE 8

High and Low Type The trends are quite similar in both types 1 on 4 was

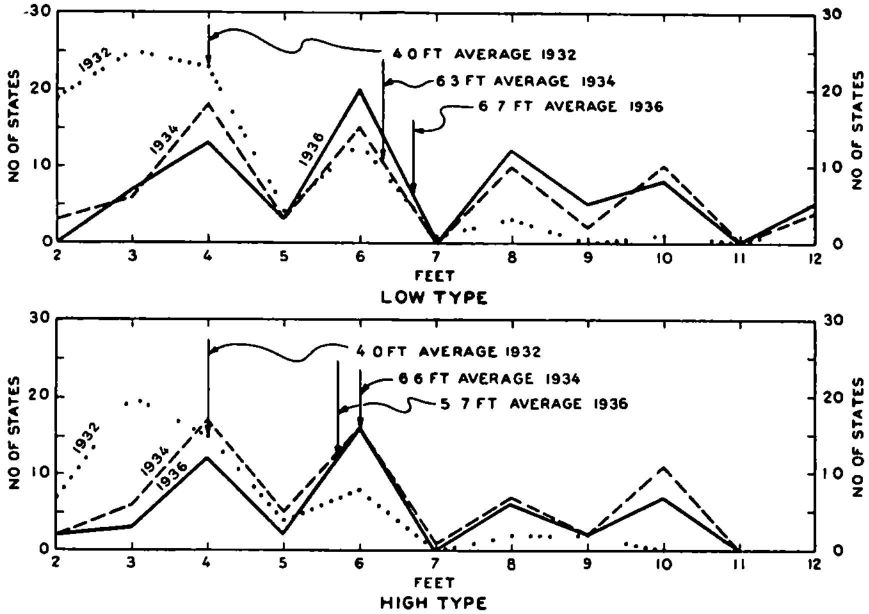


Figure 7 Trend in Widths of Gutter Slopes Used on Roads and Streets Under State Control

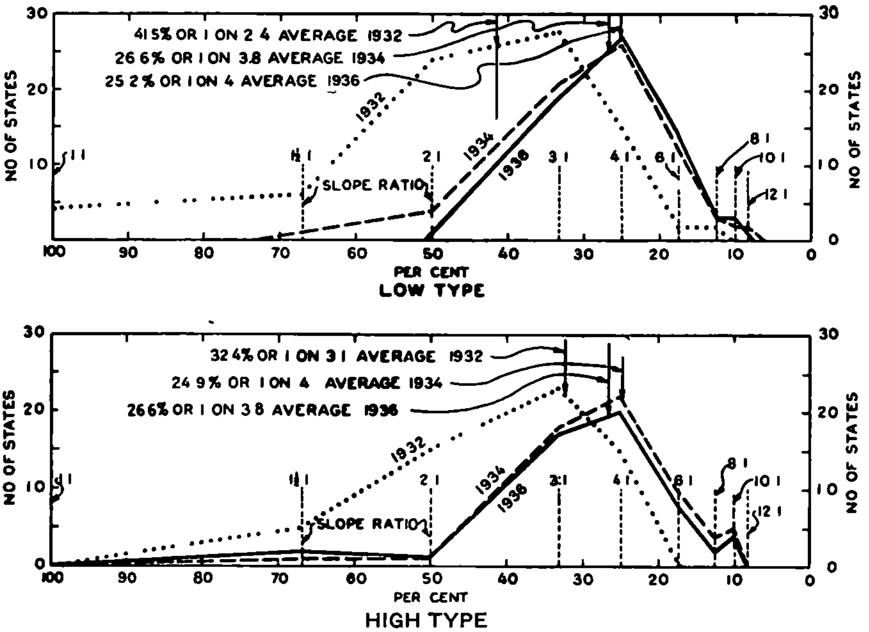


Figure 8. Trend in Percent or Ratios of Slopes of Gutters Used on Roads and Streets Under State Control

the average slope ratio used in 1936, although there is a definite indicated trend toward 1 on 6, and sometimes 1 on 10 slope Since erosion takes place on bare slopes 15 percent (1 on 6½) or steeper, the trend toward the use of slopes of 1 on 6 or flatter is logical

TREND IN WIDTH OF GUTTER SECTIONS, FIGURE 10

High and Low Type The base of section of gutter is similar in both types While zero width has been generally used, there is a definite trend toward

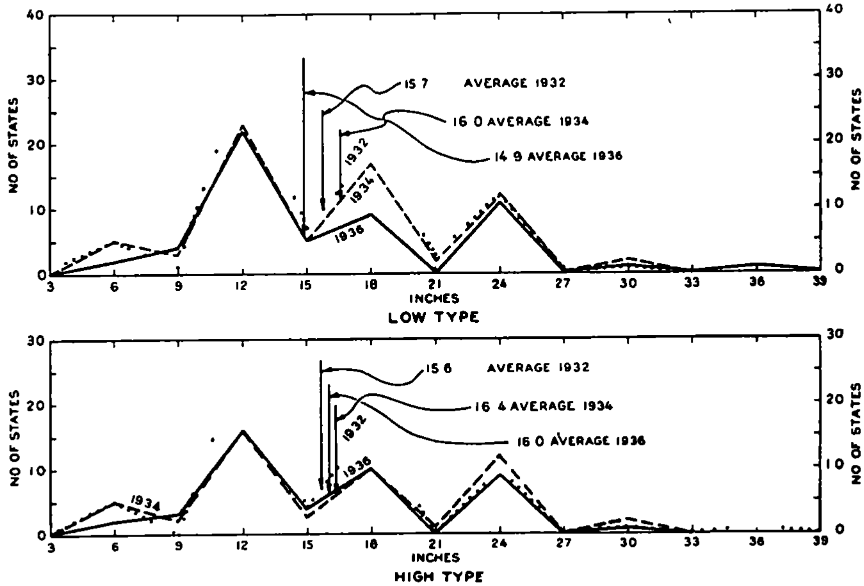


Figure 9 Trend in Minimum Depths of Gutters Used on Roads and Streets Under State Control

TREND IN MINIMUM DEPTHS OF GUTTERS, FIGURE 9

High and Low Type The average depths of gutters measured vertically below edges of shoulders is between 15 and 16 in A 12-in minimum is most commonly used, with 18 and 24 in about next in order equally The trend is toward more uniformity in gutter depth standards, in both types

Assuming a 10-ft shoulder at ½ in per ft fall from the edge of pavement, the 5-in total pitch plus 12-in minimum gutter depth, would furnish an effective depth of 17 in If the assumed crown of 1 in were added, this would make a total of 18-in gutter depth below the profile center line grade of road

effective widening to approximately 6 ft There is a definite trend also toward the rounding of gutters, making the gutter section safer and more attractive

TREND IN OFFSET WIDTHS FOR GUTTER DRAINAGE, FIGURE 11

Low Type Offset distance measured from center line to outer backslope of gutter at point level with edge of shoulder

The average offset width used in 1932 was 18 7 ft This was increased to an average of 23 4 ft in 1934, and further increased in 1936 to 23 8 ft A present trend is indicated toward the wider 30- and 32-ft figures, along with 25- and 28-ft offset distances

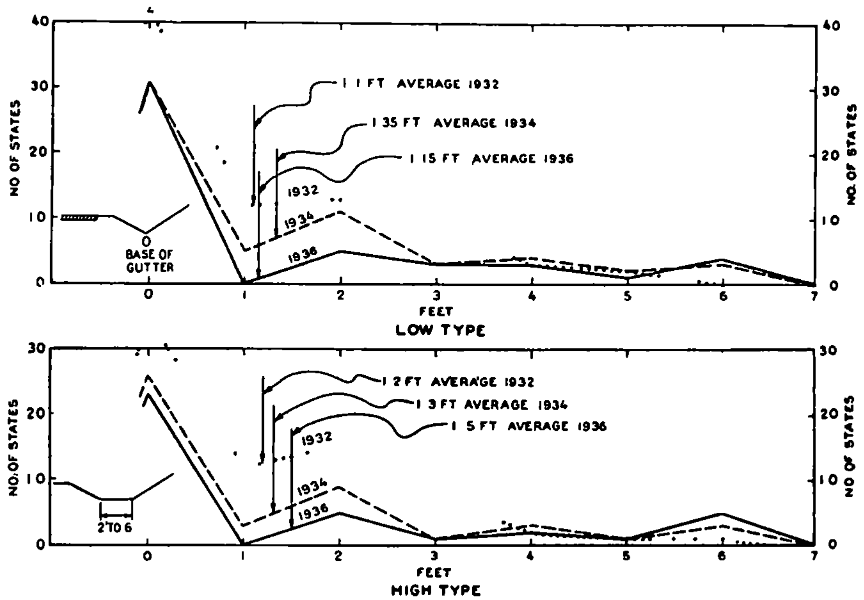


Figure 10 Trend in Widths of Gutters Used on Roads and Streets Under State Control

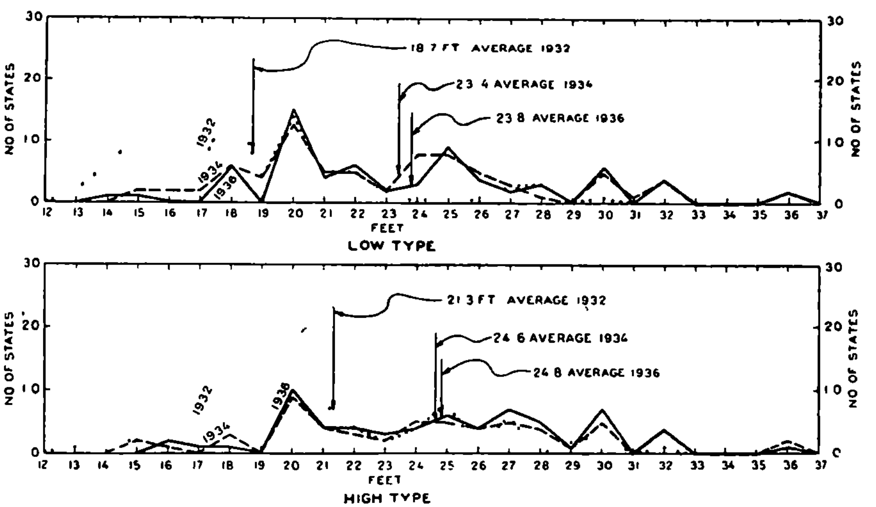


Figure 11 Trend in Offset Distances for Gutter Drainage Used on Roads and Streets Under State Control

High Type The 21 3-ft average for 1932 increased to 24 6 ft in 1934, and to a slightly higher average of 24 8 ft in 1936 The 30- and 32-ft trends apply also to this type, along with 27 and 28 ft

These dimensions should be of interest as general controls for tree plantings, pole line clearances, and so on

TREND IN PERCENT OR RATIOS OF SLOPES FOR EARTH CUTS, FIGURE 12

Low Type The 1932 slope of 83 percent (1 on 1 2) average had been reduced

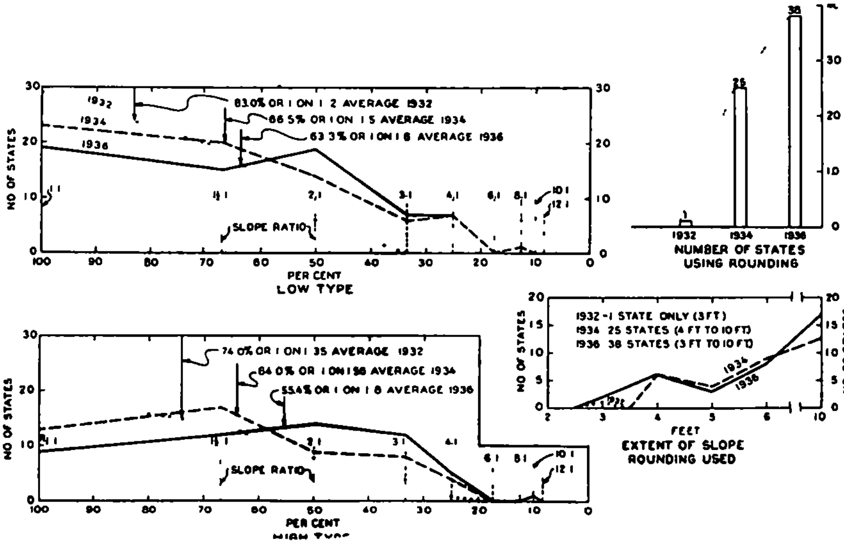


Figure 12 Trend in Percent or Ratios of Slopes for Earth Cuts Used on Roads and Streets Under State Control

20 percent by 1936 when the average was 63 3 percent or 1 on 1 6

High Type The 74 percent slope (1 on 1 3) average in 1932 was similarly reduced about 20 percent or 55 4 percent (1 on 1 8) average in 1936

In both types, while 1 on 2 slopes are most used, there are distinct trends toward the use of 1 on 3 and 1 on 4 0

Slope Rounding has also increased in extent along the flattening of slopes· In 1932, a rounding of 3 ft only in one State was used By 1934, 25 States were using rounding, and 38 States in 1936 indicated the rapid changes taking place These latter varied the rounding from about 4- to 10-ft radius or limit

Liberal rounding of slopes is emphasized

Also more attention needs be given the berm ditches at tops of cuts

TREND IN PERCENT OR RATIOS OF SLOPES FOR EARTH FILLS, FIGURE 13

High and Low Type The averages in both types are similar, from about 54 percent in 1932 through a 45 percent average to a 42 percent average in 1936 There was a relative increase in the use of 1 on 4 slopes in the latter year The use also of a few 1 on 6 and 1 on 10 slopes in 1936 indicates the possibility of flattened slopes on light fill sections

SUMMARY

1 This survey of current cross-section practices as they relate to highway landscape development outlines some of the recent adjustments made in highway design leading toward development of the

whole right of way as a unit in relation to its surroundings

2 In a typical composite section for new construction (Fig 14) based on the averages for 1932, 1934, and 1936, with a forecast of trends for 1938 and 1940, we find that the changes in highway improvement which may appear relatively slow in taking place from year to year, are nevertheless surely in progress, and when looked at in retrospect, the advance in cross-section design has shown a gradual but continuous adjustment over the period covered in this survey

3 In 1932, the slopes were steep with sharp angular intersections

4 In 1934, there is observed the first influence of the roadside improvement demonstration program upon regular highway construction Some flattening of earth slopes and some slight rounding of tops of cut slopes are indicative of the change

5 In 1936, greater flattening of slopes and more generous rounding are in evidence Right of way is typically 100 ft wide Sidewalks are beginning to appear Rounding of gutter slopes also is now noticed

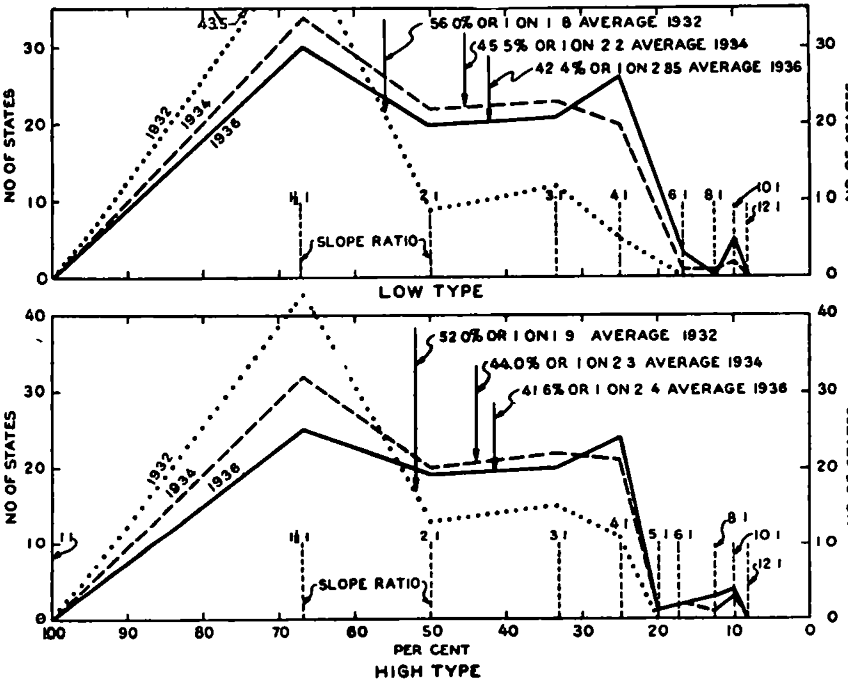


Figure 13 Trend in Percent or Ratio of Slopes for Earth Fills Used on Roads and Streets Under State Control

6 In 1938, the indicated trends point toward the following as the developing picture of tomorrow's highway, showing the stimulating effect of the demonstrated importance of the application of landscape principles in highway cross-section development as brought out in the roadside improvement program among the individual States 22-ft surfacing (2 11-ft lanes), 10-ft shoulders rounded and grassed, broad and shallow rounded gutters, easy slopes for erosion control with tops liberally rounded, wide right of way,

sidewalks for pedestrians, roadside control in the form of building set back lines and rural zoning, ultimate plan for development provided for a long term permanent improvement

7 Safety, Utility and Appearance will be served by a balanced relationship of all the parts of the highway cross-section 1 The traffic area, 2 the roadside (public) area, and 3 the roadside (private land) area The landscape development of highways will assume its proper place in the programs of the future, in the problem of coordinating these areas

8 The construction approach to the highway problem will be supplemented with a careful study of uses of each part making up the whole functioning highway, with a view to meeting the present as well as future demands of traffic, and at the same time providing the highest possible degree of safety This will include the collection of data on the use of the parkway type, the freeway type, and the highway type, two-lane, three-lane, and four-lane highways, the proper shoulder width, and other related matters which affect the safety of the vehicle, whether in motion or parked off the pavement, such as mail box turnouts, bus stop turnouts especially for school bus routes, safety turnouts, accelerating and decelerating lanes at intersection points, etc

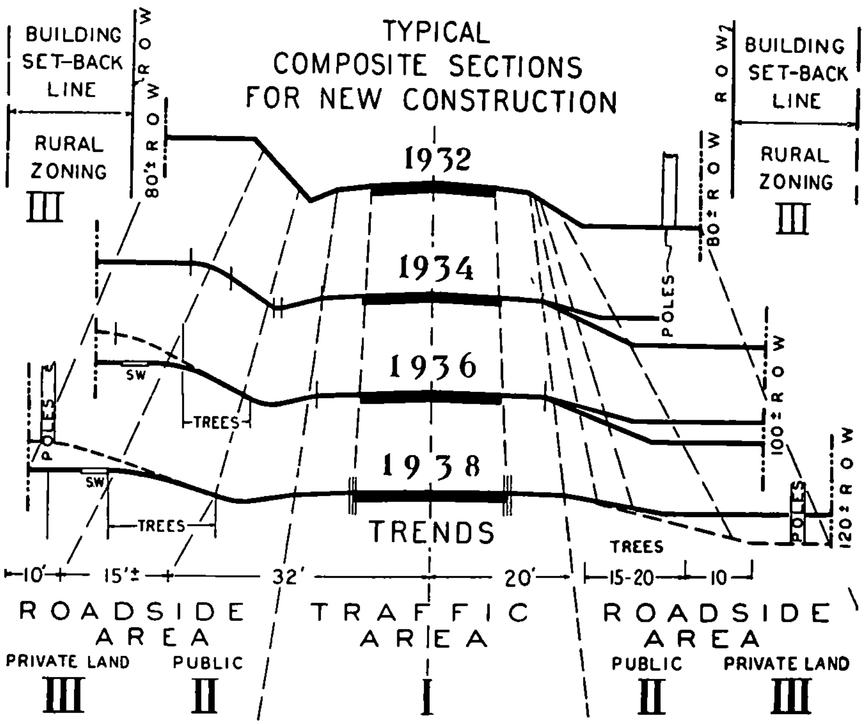


Figure 14

9 This indicated "use" pattern may suggest a "standard" by which some greater coordination and uniformity can be obtained in the future typical highway cross-section These trends are indicative of the growing emphasis now placed on the application of the princi-

ples of landscape architecture in the development of America's highways

10 Knowledge of the general trends in the direction of a wider application of the principles of landscape architecture to typical highway problems is of value to public officials who plan State or local programs of highway development and formulate maintenance policies

Facts regarding the roadside features of highway design are of interest to all individuals and organizations To the highway designer, the safety aspects of landscape development are of particular interest at this time

From these evolving "patterns," it is indicated that the best appearing road is *safest,* and *the safest road is attractive in appearance*

DISCUSSION ON HIGHWAY CROSS-SECTIONS

MR W W MACK, *Delaware State Highway Department* I would like to ask about the planting of trees along the highway In Delaware we have had several serious accidents this year in which four people were killed by cars running into trees along the highway There was some public agitation directed toward discontinuance of such planting

MR SIMONSON That is a very good question

The typical composite sections for new construction, Figure 14 of the paper show trees to be kept 5 to 9 ft from the edge of the shoulder In general possibly 80 percent of the trees should be kept beyond that minimum clearance line Only an occasional tree or perhaps from 10 to 20 percent of the total number of trees planted should come out at logical points in the zone 5 to 9 ft from the edge of shoulder The desirability of this of course can be demonstrated in the progressive widening of those sections where tree planting may be done on an older narrow section because trees require time to mature and therefore should be placed in as permanent a position as may be foreseen outside of any possible future widening of the highway If a vertical line is projected down from the typical 1932 widths (Fig 14) it will be seen that it is encroaching upon the pavement within a period of only a few years That emphasizes the importance of careful planning in establishing a policy for the control of trees which are more or less permanent features of the highways It also emphasizes the need for highway departments to prepare complete sections that include the whole highway right of way problem showing in a general way those policies with regard to trees, poles, etc , that they have adopted as a guiding control for a safer highway development

HIGHWAY DESIGN ITS RELATION TO LANDSCAPE OBJECTIVES

By A R Nichols

Consultant Landscape Architect, Department of Highways, State of Minnesota

SYNOPSIS

Highway design has been concerned essentially with engineering objectives in the scientific solution of grading, paving, alignment, profile, cross section, drainage and maintenance problems These were prerequisites of good construction but were incomplete in their relation to the proper blending of the construction into the natural landscape Highway design in its broadest application will not only meet the requirements of safety and utility but will aim for ideals in the realm of landscape objectives and incorporate beauty in the completed structure The landscape objectives may be summarized as follows

1 The utilization of existing scenic advantages in the determination of a proposed route intended largely for pleasure traffic
2 The harmonizing of construction with natural topography by coordinating the work of the engineer with the landscape architect in all stages of reconnaissance, planning and construction
3 The conservation of existing vegetation and trees as far as is consistent with utilitarian requirements
4 The planting of new material primarily as a contributing agent to control erosion and to accomplish a natural transition between construction and nature
5 The creation of featural developments such as outlooks, concourses, parking spaces, picnic areas, historical marker sites and similar strategic areas where the public can stop for rest and enjoyment
6 The promotion of liberal right of way for the elimination of old scars on existing roads, the greater ease of blending construction into the natural topography on both old and new roads and for the protection of the roadside in case of future widening
7 The encouragement of separation of commercial from pleasure motor traffic, thus permitting parkway emphasis and greater latitude in the design of the pleasure route
8 The attainment of zoning for the better control, regulation and restriction of billboards and commercial structures along the highway

Such objectives logically go hand in hand with engineering objectives and will reduce drastic departures from the natural lay of the land and automatically reduce the problems of erosion and of maintenance The total of all objectives is a balance of safety, good construction, economical maintenance and natural beauty

Every human activity has certain definite objectives These objectives may be casual and incidental, or they may be specific and fundamental They may be limited in their scope or they may be broad and comprehensive in their influence In every case, the higher the ideals of achievement, the broader have been the objectives that have directed the mind and the hand in the process

The engineering profession has much to be proud of in the accomplishment of objectives essential to the program of highway development It is natural that the objectives have been largely utilitarian as creations serving the needs of man have always been the main purpose of the engineer The coming of the automobile in an age closely following the development and expansion of our railway system resulted in the application of the general principles of railway engineering to the design of our highways The requirements for road conditions adapted to the automobile were so pressing that the immediate and laudable objective of design was to furnish the public a better means of travel and communication

The engineering objectives, therefore, were logically the scientific solutions of the problems of grading, paving, alignment, profile, cross section, drainage and maintenance These were the essential prerequisites of good construction Tremendous achievements were made and the later recognition of additional objectives in design cannot in any way belittle the credit due the engineer in his attainment of the basic objectives Good roads were provided, increased speed made possible and a definite contribution made to commerce as well as to convenience of access

The program progressed rapidly, and with the completed work there developed problems related to the roadside which temporarily had seemed incidental Construction scars, ugly fills and backslopes and the destruction of natural vegetation, however, created problems of erosion and of maintenance as well as marring the beauty of the roadside The general results all over the country were sufficient evidence that the previously determined objectives were inadequate to a complete fulfillment of a satisfying design There had been lacking any definite landscape objectives in the program that considered sufficiently the roadside problems in their practical and esthetic bearing upon construction and maintenance It became apparent that the two objectives, the fulfillment of utilitarian requirements and the attainment of esthetic values were not divergent but in reality were closely related, and that the landscape design must be an integral part of the road design in all its phases

It is important, therefore, that the objectives be studied in order that a clearer understanding may be had of the basic principles and that coordination of effort may lead to a broad and comprehensive solution of a problem

RESTRICTED OBJECTIVES

Due to the large amount of follow up work necessary on old routes and a recognition that these routes were ugly in their unnatural sloping and resulting erosion, the landscape objective was at first thought of as a dressing up and correction of old scars This suggested the unfortunate term "Roadside Beautification" which is fundamentally at variance with the true purpose of highway design Even the later term "Roadside Development" while expressing a more logical objective, infers a restricted scope of treatment subsequent to construction, and in a defined area between the paved road and the right of way lines Such an objective is one of "Applied Art" rather than one of "Landscape Design "

There has been too great a tendency in all our states to consider that the landscape objectives lie within a fixed right of way From a strictly construction and maintenance point of view this is correct, and roadside development has a most important field of endeavor to remodel and improve the roadsides of the older routes

BROADER LANDSCAPE OBJECTIVES

Highway design, in its broadest terms, however, has landscape objectives that are definitely related to the proper alignment, gradient and cross section—thus partly determining the right of way itself and its relation to the area lying beyond the limits of right of way It is the recognition of beauty in the country beyond the right of way that makes the lasting impression upon the traveler He sees the method of treatment of the roadside, but the vision carries beyond to the larger aspect of the landscape Landscape design is more concerned with the cooperative endeavor of fitting construction into this larger picture than it is with the remedial work within predetermined limits of right of way

Highway design should, therefore, include certain definite landscape objectives which would bring construction more closely into harmony with the total terrain and which would contribute to a development more complete, more satisfying and more far-reaching in its service to the use and enjoyment of the public The question immediately arises First, what are these landscape objectives? Second, how can the highway design bring about their realization?

UTILIZE SCENIC BEAUTY

Our country abounds in a wealth of magnificent scenery of great variety The public is giving increased attention to these natural assets, and it is essentially a part of the highway program to plan for convenient access to points of interest and areas of recreation The present development and popularity of our state parks and the rapidly growing tourist interests of the country have tremendously increased the amount of pleasure traffic on our highways Already reaching huge proportions, I believe that the pleasure traffic will continue to grow especially in consideration of the shorter working hours and the increase in leisure time made available to the public at large The lure of the woods and mountains, the lakes and the seashore, the meadow and pasture, is common to all It is highway design that not only determines the means of access to these places of human enjoyment, but also determines whether that route of traverse shall be attractive or unattractive It should be one of the definite objectives in highway design to take every advantage of scenic beauty and accentuate its importance in every possible way This objective immediately involves a careful coordination of work of the engineer and the landscape architect during preliminary reconnaissance, in order that the most practical general route may be determined, taking advantage of the specific scenic factors that would enhance the ultimate solution The utilitarian objective might logically influence the design to determine a route somewhat shorter in distance or possibly of less difficult construction However, the landscape objective might justify a somewhat longer route or a more difficult construction, if the scenic values seemed to the engineer and the landscape architect to offset the financial difference This objective dictates a careful study of such opportunities as locating the route so as to skirt some beautiful lakeshore or rising to some elevation where a commanding view would make a lasting impression upon the traveling public It must be borne in mind also that these scenic values are virtually permanent values, whereas the structural values in time may diminish

To fully utilize existing scenic values in the initial stages of reconnaissance and highway design is the first and most important landscape objective

HARMONIZE CONSTRUCTION WITH NATURAL TOPOGRAPHY

The second landscape objective is that of "Harmonizing Construction with Natural Topography" It is this objective that is dependent upon the principles of landscape design as the construction itself is dependent upon road design Again, the objectives should not be divergent, for if there is proper coordination of effort, the best construction in its broadest application will embody beauty in its achievement and ultimate economy of maintenance Adequate right of way may assist in partially correcting constructional departures from natural topography, but if the most satisfactory transition from construction to natural landscape is to be accomplished, it must be definitely provided for in the highway design The objective should be to pre-

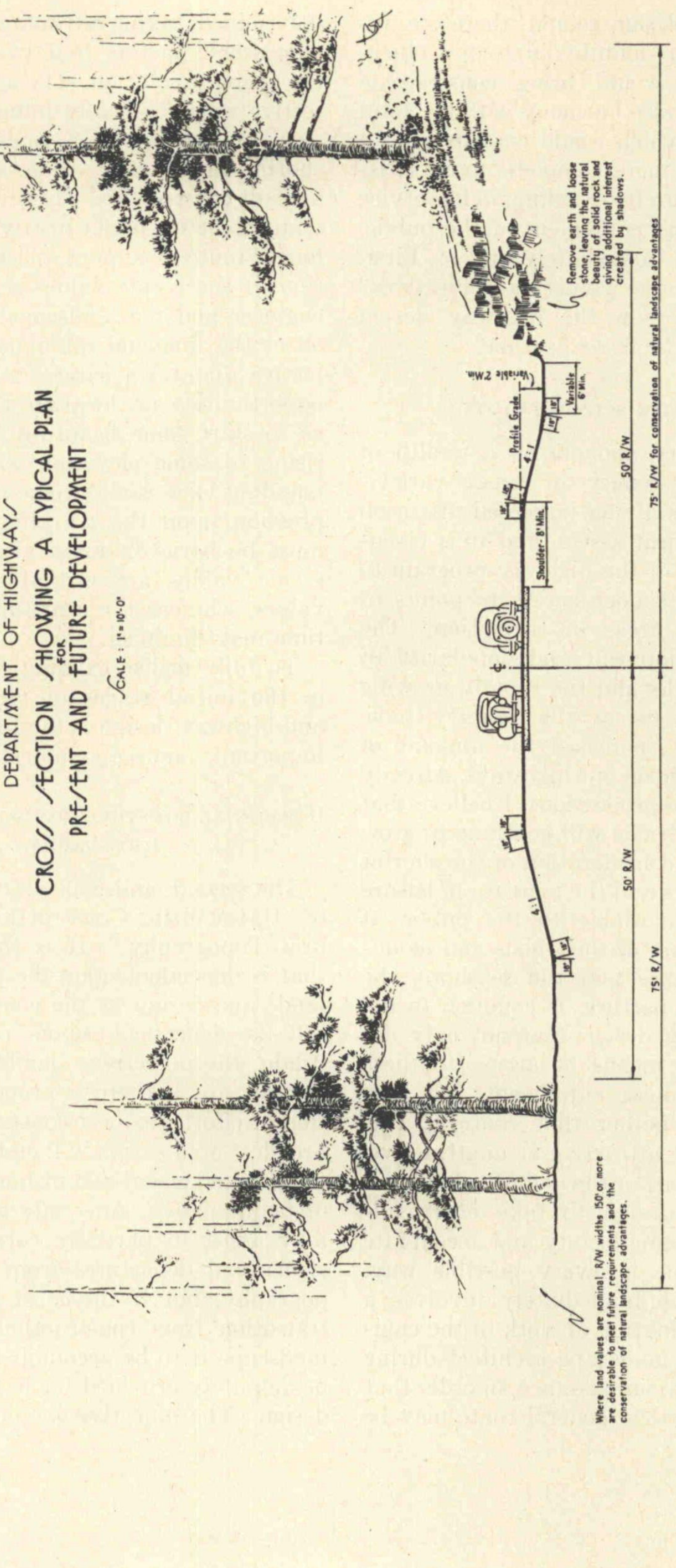

STATE OF MINNESOTA
DEPARTMENT OF HIGHWAYS
CROSS SECTION SHOWING A TYPICAL PLAN
FOR
PRESENT AND FUTURE DEVELOPMENT
SCALE: 1" = 10'-0"
Remove earth and loose stone, leaving the natural beauty of solid rock and giving additional interest created by shadows.
Variable 2' Min.
Profile Grade
4:1
18"
18"
Variable 6' Min.
Shoulder - 8' Min.
4:1
50' R/W
50' R/W
75' R/W for conservation of natural landscape advantages
75' R/W
Where land values are nominal, R/W widths 150' or more are desirable to meet future requirements and the conservation of natural landscape advantages.

Figure 1

serve and restore the natural conditions by means of as simple and as graceful a grading and planting program as is possible. This simplicity and naturalness comes only when the original alignment and gradient has been studied and planned with this landscape value as a definite objective.

Mr. Thomas H. MacDonald, Chief of the Bureau of Public Roads, in a recent, "Review of Progress," makes the following statement: "For harmony of line, roadways should follow the regular lay of the land as far as is consistent with safety and utilitarian requirements." Here is a statement, in my opinion, that implies the underlying reason for many mistakes of the past and points to the ideal objectives in highway design. I believe you will all agree with me that the most glaring examples of artificiality in highway construction can be attributed to the fact that the alignment and profile were not sufficiently closely related to the regular lay of the land. The safety and utilitarian requirements were carefully planned, but the study of how closely the line and profile might have followed the lay of the land was somewhat neglected. The inevitable result has been that of ugly cuts, high fills and an increased area of destruction of natural vegetation. Landscape design objectives must be combined with engineering objectives in the determination of alignment, profile and cross section. Here is where "Design for Safety and Utility" can logically harmonize with "Design for Beauty" in studying the principles of perspective as applying to horizontal curvature, vertical profile, and proportional relation of tangent and curvature.

In the *design of the cross section,* the engineering objectives and the landscape objectives should easily be harmonized for the satisfactory accomplishment of the ideals of safety and economical maintenance which will automatically fulfill certain landscape objectives. The last five or six years have witnessed a marked change in the grading of fillslopes, ditches, and backslopes, largely due to the continued emphasis placed by the Bureau of Public Roads upon the relation of these features to safety and maintenance. The wider shoulders, the flatter fillslopes and backslopes have demonstrated their value with respect to erosion control by the greater facility of restoring natural ground cover. There is still, however, the tendency in original construction design and supervision to neglect the essential factor of fullest conservation of topsoil and its utilization on the newly graded slopes. The report of the sub-committee on erosion of the Joint Committee on Roadside Development makes the following important statement:

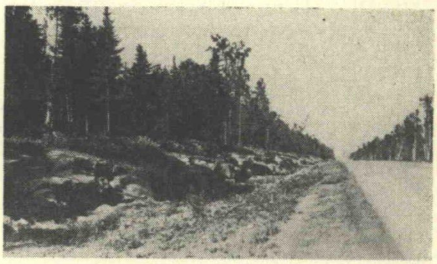


Figure 2. Photograph of Section Depicted in Figure 1

"On shoulders, ditch areas and slopes a sterile subsoil has usually resulted from grading construction or maintenance operations where no thought was given to the saving of topsoil. Satisfactory vegetative cover cannot be established by natural or artificial methods without a deep soil capable of supporting plant growth." This committee offers, therefore, as a basic objective of both roadside development and soil erosion research that "on highway lands, design must be improved to provide a satisfactory surface upon which vegetation can be established as a final stage of controlling erosion."

All of these practical considerations in the design of the cross section have a definite bearing upon the esthetic objective. The grading of fillslopes and backslopes not steeper than three to one and the rounding of the tops and toes of slopes and the restoration of ground cover not only assist materially in accomplishing the control of erosion and more economical maintenance, but also brings the construction more closely into harmony with the natural surroundings.

PLANTING; A CONTRIBUTING AGENT

The utilitarian objectives, particularly those of erosion control and economy of

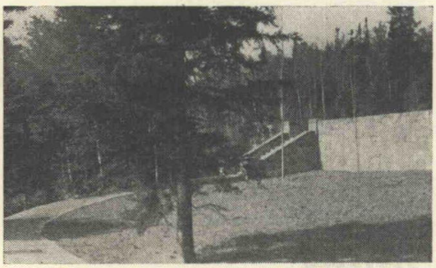


Figure 3

maintenance, are greatly aided in their accomplishment by the use of plant material properly selected and arranged. It is self evident that if we are to bring about the landscape objective of harmony with nature, this material must be studied in the light of the climate of the locality, the existing soil conditions, and the natural habitat of the plants as well as their adaptability and suitability to highway use. Planting design aims to stabilize the regraded surface of the construction area and to restore a natural blending of the structural elements with the undisturbed environment. However, planting is merely a means to an end and should not be considered in itself as a landscape objective. It may be to give shade to a barren waste, it may be to screen objectionable views or to frame an attractive vista. It may be to assist in the control of erosion on a slope or it may be to restore a natural background. However, in any case, it must be as a means or contributing agent toward the main objective of *harmonizing the construction with natural surroundings*. For this reason, planting can only function properly when applied to a naturally graded area. Too often, we have considered planting as the real landscape objective with the result that confusion and discord have been the outcome instead of harmony. Fundamental design in the determination of location, the proper alignment, profile and section is the real governing factor, with planting as an embellishment and refinement to the more essential agencies.

CREATIVE DESIGN AT POINTS OF SCENIC INTEREST

It is not sufficient in the entire development merely to "Utilize the Natural Scenery" in its application to proposed alignment and profile and to "Harmonize the Construction with Natural Topography." These are the most essential objectives but to fully accomplish their purpose there must be added definite development of architectural, engineering and horticultural accessories.

Closely linked with the design of the roadway is the desirability of taking advantage of the outstanding opportunities for the creation of concourses, outlooks, picnic areas and parking spaces wherever such development is justified by scenic interest and public use. Up to the present time, the city or the town has been the individual point or focus toward which we have directed our line of traffic. Highway planning has been solely concerned with the utilitarian objective of the facility and speed connecting these individual centers. With the increasing tourist travel and the importance of the scenic values, we should recognize as a desirable part of the highway design, the landscape objective of providing stopping places for full appreciation of areas of commanding view, and for suitable rest and recreation. No longer is the problem merely one of getting from "here" to "there." It is rather what points of interest can be developed that will add to the enjoyment of the public in traveling between "here" and "there." Wherever emphasis has been given to this type of work and architectural simplicity has dominated the design of outlooks, concourses and parking spaces adjacent to picnic areas and historical markers, there has been invariably most favorable comment by the public. Such featural development relieves the monotony of the continuous travel, adds interest to the total development program and contributes to the broader application of design in its landscape objective. Additional right of way to make possible these developments permits cars to swing off the main traveled road thus increasing safety at these strategic points. Much of this structural work in the building of retaining walls for outlooks and concourses has been accomplished in many states through the cooperation of the National Park Service and through the various Relief Agencies. I urge the enlargement of this field of study as a definite landscape objective in the complete highway design aiming to give to the public the fullest degree of use and enjoyment.

OBJECTIVES RELATED TO RIGHT OF WAY

The ideals of practically every landscape objective are often defeated by an existing narrow right of way or by a right of way on a proposed route which has not been determined as to its description and its width by a careful study of the esthetic requirements as well as the safety and utilitarian requirements. Artificiality and ugly scars leading to erosion problems and high maintenance costs might well be said to be synonymous with narrow right of way. It is my opinion, that *all attempts at erosion control and landscape development under the limitations of a narrow right of way are, at best, merely a partial and inadequate attainment of the landscape objective.* These attempts do not reach the underlying cause of the trouble and are, therefore, not fundamental in character. Upon the open road, it is far more essential and fundamental to correct an artificial cut or fill by flattening the slopes than to sod or plant the steep slopes. Wherever possible on old align-

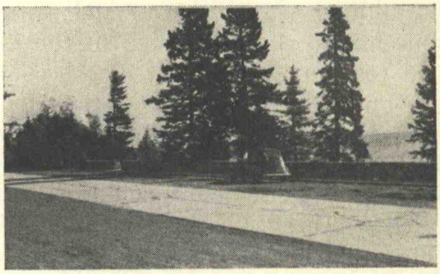


Figure 4

ment, the design for roadside development should direct its attention to the fundamental principle involved of proper grading, rather than perpetuate artificiality by superficial methods. This involves adequate right of way for a satisfactory solution of the desired landscape objective. Acquisition of added width or the procurement of slope easement should be urged, if we are to attain the landscape objective of harmonizing construction with natural topography.

PARKWAY EMPHASIS

One of the most encouraging factors in the attainment of many of these landscape objectives is the present trend toward a separation of heavy commercial traffic from the pleasure motor traffic It must be admitted that many of the ideals desired in alignment, profile and cross section have not the same flexibility, when applied to the commercial route as when applied to the pleasure traffic route There is a vast opportunity for landscape development on the parkways or scenic routes, the divided roadways, the freeways and all the other forms of highways which give greater breadth of development to the handling of traffic The Mt Vernon Memorial Highway, The Westchester County Parkway, The Virginia Skyline Drive, and the Blue Ridge Parkway are all examples of the recognition of the importance of landscape objectives in meeting the needs of pleasure traffic, yet fulfilling the utilitarian and safety requirements In the design of the divided roadway which will now become more and more in evidence, the landscape objectives must conform to those same principles already enunciated and make the entire structure one of natural unified appearance rather than two distinct developments divided by an island of unrelated height or depth The total cross section must be such that both roadways seem to fit the natural setting This can be accomplished only when the island treatment which divides traffic in reality unites the development of the entire right of way Simplicity of grading and simplicity of planting should dominate the island treatment

ZONING

A paper dealing with highway design and its relation to landscape objectives would be incomplete without reference to that most important field—highway zoning Other attainments from objectives in the esthetic program can be completely marred by the conspicuous presence of billboards, signs and undesirable types of industries Much study, research and determination of policy lies ahead of us, with respect to this part of highway growth and development The subject of zoning is too large to more than mention in this discussion It reaches into the problems of regulation of highway uses and restrictions in the interest of safety and protection of values as well as into the question of roadside appearance These considerations are not essentially those pertaining to highway design but rather to state planning and county zoning with all the intricate questions of legality of discretionary powers based upon esthetic considerations Nevertheless, the ultimate elimination of billboards, snipe signs and the control, regulation and restriction of commercial structures along the highway is a very definite landscape objective One of the best examples in the partial attainment of this objective is that of California, where highway zoning as a part of county zoning has enabled the State in some eight or more counties to limit outdoor advertizing to the business centers of built up communities and to establish small business districts at properly located intervals along the highway, through the rural districts No outdoor advertizing is permitted between these small business districts New Jersey through its State Planning Board, Massachusetts and many other States are making constant advance toward the goal of highway zoning

SUMMARY

May we, therefore, summarize these landscape objectives as follows

1 The utilization of existing scenic advantages in the determination of a proposed route intended largely for pleasure traffic
2 The harmonizing of construction with natural topography by coordinating the work of the engineer with the

landscape architect in all stages of reconnaissance, planning and construction

3 The conservation of existing vegetation and trees as far as is consistent with utilitarian requirements

4 The planting of new material primarily as a contributing agent to control erosion and to accomplish a natural transition between construction and nature

5 The creation of featural development such as outlooks, concourses, parking spaces, picnic areas, historical marker sites and similar strategic areas where the public can stop for rest and enjoyment

6 The promotion of liberal right of way for the elimination of old scars on existing roads, the greater ease of blending construction into the natural topography on both old and new roads and for the protection of the roadside in case of future widening

7 The encouragment of separation of commercial from pleasure motor traffic, thus permitting parkway emphasis and greater latitude in the design of the pleasure route

8 The attainment of zoning for the better control, regulation and restriction of billboards and commercial structures along the highway

The landscape objectives, thus, are those which will unite logically with the engineering objectives in giving to the complete work a beauty of structural achievement in a natural setting Such objectives will reduce drastic departures from the natural lay of the land and will automatically reduce the problems of erosion and of maintenance

CONCLUSION

Objectives determine the direction in which we are progressing Design should progress as a smoothly organized piece of machinery carried forward on four well balanced wheels The first of these wheels is Safety, the second is Good Construction, the third is Economical Maintenance, and the fourth is Natural Beauty Each wheel has its individual function to perform and must fulfill a balanced relationship with every other wheel Unified progress cannot result, if there is an unequal brake action or improper freedom for each wheel to function in balance with the others

The Highway Research Board is giving careful study to all the essential factors involved in a complete highway structure Such completeness is now recognized as resultant from design that not only fulfills the utilitarian and safety requirements, but that also aims for ideals in the realm of landscape objectives and incorporates beauty in its structure May there be a closer cooperation of engineer and landscape architect in the study of all the objectives of highway design, in order that there will be continual progress toward the ideals of complete perfection

Mr Daniel H Burnham has well said

"Make no little plans, they have no magic to stir men's blood Make big plans, aim high in hope and in work remembering that a noble, logical diagram once recorded will never die, but long after we are gone will be a living thing asserting itself with ever growing insistence Let your watch word be Order and your beacon Beauty"

DISCUSSION ON DESIGN AND LANDSCAPE OBJECTIVES

MR O L KIPP, *Minnesota Highway Department* While there is no question about the desirability of roadside development projects to correct mistakes previously made on construction where such projects will not be changed by demands for further improvement to accommodate increasing traffic needs, yet the greatest benefit can be obtained by carrying out the principles which Mr Nichols has outlined on all new construction It is possible, of course, on new construction to obtain much more pleasing effects with little if any increased cost than can be obtained in attempts to correct or embellish previous construction

MR FRED LAVIS, *Consulting Engineer* One phase of this which occurs to me is this A little while ago I was driving over a highway in an extremely hilly country It has been located for miles and miles on a straight line, and it occurred to me in driving over it that probably it would have been a better, safer and more attractive road if some curvature had been introduced into it It may be heresy to say, with some curvature, when a straight line is possible, but I am not so sure about that I am an old-time engineer but I believe in that particular case a fair amount of light curvature would have made it a better road to drive over with very little difference in gradients Also a certain amount of curvature would have saved on construction cost

MR W H SIMONSON, *U S Bureau of Public Roads* In analyzing the construction of a highway the engineer cannot limit his analysis to just the construction point of view alone in cold facts and figures He must also relate the elements of construction to the use or function that the road will serve If the human element is the largest factor in the safety triangle, the more necessary it is that we approach our construction and design problems from the standpoint of the human equation We will never have a complete and safe highway until that is done

REPORT OF DEPARTMENT OF MATERIALS AND CONSTRUCTION

C H SCHOLER, *Chairman*

REPORT OF COMMITTEE ON METHODS OF HANDLING AND PLACING CONCRETE

SYMPOSIUM ON VIBRATION OF PAVEMENT CONCRETE

OPENING DISCUSSION

F H JACKSON, *Chairman*

Senior Engineer of Tests

U S Bureau of Public Roads

This year the Project Committee on Methods of Handling and Placing Concrete is sponsoring the presentation of four reports on the subject of vibration One of these covers French practice, the other three the results of tests and observation on the application of vibration to pavement construction in three states,—Wisconsin, Kansas and Illinois It is hoped that the presentation of these reports and the discussion following will stimulate interest in the possibilities of this construction practice as a means of improving the quality of pavement concrete

It is a curious fact that, in spite of the almost uniformly encouraging reports of field experiments which have been received since the modern method of high frequency vibration was first proposed in 1930, there has been no general acceptance of the process by the various states This may be due in part to a natural inertia which tends to resist changes in construction practice as well as to a feeling on the part of engineers that the advantages so far indicated are not of sufficient magnitude to balance the increased equipment charge There may also be a feeling that production may be slowed down sufficiently to result in an increase in construction costs It is true that the increase in strength, in terms of which the quality of pavement is usually measured, is not great, only about 10 percent for a given cement content However, it must be remembered that experience to date, both in the laboratory and in the field, indicates that vibration imparts other desirable qualities which, although not so easily evaluated as strength, are fully as important The ability to handle drier, harsher mixes than are possible when the usual surface screeding methods of finishing are employed, should result in denser and consequently more durable concrete

The matter of surface durability of pavement concrete is of prime importance There is no question but that modern methods of finishing, which emphasize smoothness above everything else, tend to produce a surface layer of weak, porous mortar which becomes an easy prey to weathering or to the action of salts used for ice removal The tendency to use a highly oversanded mix in order to insure maximum workability should also be discouraged Vibration should tend to correct these evils to a certain extent by making it possible to use less sand and water and more coarse aggregate per unit volume of cement This reduces the excess mortar left for finishing and, moreover, the mortar which does come to the top contains less sand and water in proportion to its cement content and consequently must be more durable

In virtually all of the reports of field experience with vibration so far made, the point is emphasized that the harshest and driest mix which may be used is controlled not by the vibratory equipment but by other units in the mixing, placing and finishing operation The possibilities of high frequency vibration are distinctly limited so long as we insist on using mixer buckets and finishing equipment designed for plastic mixtures The problem calls for a new approach We must study all of the operations involved in the manufacture and handling of the concrete with a view to so designing the equipment that the potentially high density and strength of the very dry harsh mixes may be realized in construction This would appear to be a problem well worth the attention of the Joint Project Committee on the Development of Highway Construction Equipment

It should not be inferred from the foregoing that the benefits which may be derived from the use of vibration with equipment now on the market are not of practical significance It is desired simply to point out the possibility of still further benefits which should result when the other units are re-designed to handle the type of concrete most suitable for vibration In the meantime it would seem that the states could well afford to use this new tool more generally than has been the case up to the present time The discussion this afternoon will show beyond question that the application of vibration using existing equipment is practical and economical This fact, coupled with the improvement in quality which has been demonstrated should result in a wider and more general acceptance of vibration than has been evidenced up to this time

METHODS OF VIBRATION OF CONCRETE ACCORDING TO FRENCH PRACTICE *

By J S Crandell

Professor of Highway Engineering, University of Illinois

SYNOPSIS

French practice goes in for heavy vibrating or tamping of pavement concrete Brief descriptions of several machines are given, some of which run on side rails and others rest directly upon the fresh concrete A feature of the practice described by M Fedi is a brushing machine for roughening the surface to make it skid proof by removing the fine material during the setting and hardening period M Fedi believes in use of lean mixtures with enough vibration or ramming to force the individual pieces of large size aggregate into such intimate positions that they are securely bonded to each other by a minimum amount of cement Almost no cracking and little wear was observable on ten year old pavements, in the heart of Paris, laid by these methods

Machines for the construction of concrete pavements in France may be classified into two categories

1 Those for use without side-forms, or lateral rails, where apparatus and machinery are handled and moved by hand, or are made to roll directly on the freshly laid concrete

2 Those provided with tracks or side-forms, on each side of the pavement, on which machinery is mounted

EQUIPMENT FOR TYPE 1

There are several machines that will either ram, tamp, or vibrate concrete The hand rammer is the most primitive Then we have the vibro-rammer, the effect of which is intermediate between that of ramming and of vibration The long roller is employed for hand-surfacing Some of these rollers act by means of the vibrators placed in them as well as by their weight Since they rest directly on the fresh concrete their weight aids in compaction

Another machine is called a vibro-hammer Of these we have the "vibro-pil," the "velo-rammer," and the "vibro-plat" These tamp the concrete with high frequency vibration but at a very small rate, such as from one to two tenths of a millimeter Their use assures a fairly uniform surface

The "vibro-tasseur" rams the concrete at low frequency but at a large rate, such as one to four centimeters A surface identical with that produced by the preceding apparatus is secured

There is a most satisfactory machine with multiple pneumatic rammers whose speed of tamping is adjustable This machine is mounted on large, wide rollers which sink but slightly in the fresh concrete at first, and do not sink at all after it is rammed A motor is part of the apparatus This is followed by a surfacing machine which moves transversely across the pavement on suitably ballasted big rollers A very stiff dry concrete made with large size aggregate can be satisfactorily surfaced with this machine

And then there is a patented brushing machine for roughening the surface of the concrete to make it skid proof The machine is started running after the initial set has taken place and it is kept running during the hardening of the concrete It wears away the cement paste by means of the wire brushes which are driven by the motor of the machine

* Information taken from "Les Bétons et les Revêtements Betonnés de Chaussées" by J Fedi, together with facts observed in the field by Professor Crandell

EQUIPMENT FOR TYPE 2

There are several machines that may be mounted on the side-forms or on rails on either side of the pavement Among these is the Dingler ramming machine equipped with transverse lines of mechanical rammers This machine performs in a very energetic manner It is extremely heavy and requires excellent support to do good work

The Fedi ramming machine is equipped with longitudinal lines of mechanical rammers which work and shift transversely, and then may be moved longitudinally This machine has some advantages over the Dingler since the weight may be limited by leaving off extra rammers The ramming may be as powerful as desired Any width of pavement may be rammed

The Dingler and the van Steenkist machines do surfacing work similar to that produced by the preceding one The van Steenkist type has a telescopic frame which makes it easily adaptable to different widths of pavement

The Fedi surfacing machine does its work on tracks identically as well as the machine described in Type 1 Similarly, there is a Fedi brushing machine on tracks that operates in the same manner as before described

From the illustrations and the description furnished it will be inferred that the French methods of vibrating or tamping concrete are severe This is true I have seen many a dry mix that American contractors would say is not workable finished perfectly by the French methods The rough treatment accorded concrete in the making seems to secure results that are satisfactory Fedi says that pavements laid without machinery are not of much value, and that if machinery is used it should be of such a nature that a dry, lean mix can be vibrated or tamped so as to produce a "visible mosaic surface" that provides rugosity (skid-proofness) and still is well finished To that end he uses the brushing machine to clean off the fines, thus exposing the coarse aggregate

If we think back on our American practice it will be recalled that in the Vibrolithic pavement, invented by Mr Stubbs, a course of large, extraordinarily tough, hard, dense stone was incorporated in the surface before or during the time of vibration M Fedi makes sure that such stone constitutes the large aggregate for his pavements, but he does not spread a layer over the top as did Mr Stubbs Instead he uses a wire brush of rather large diameter which revolves, brushing away the fines and exposing the large aggregate This makes a skid-proof surface It is of course essential that the large size aggregate shall be firmly held in the concrete, otherwise there will be considerable deterioration

Fedi states that there is a great difference in the hardness of the large size aggregate and the cement paste He believes in a minimum of the latter, for he seems to think that a lean concrete can be made to do better work than a rich one, if both are vibrated or tamped He would vibrate, tamp, or ram the lean mix until he forces the individual large size aggregate into such intimate positions that each piece is securely bonded to its neighbor by a minimum amount of cement He believes that steel reinforcement is preferable to increased thickness of slab when and if the subsoil is not stabilized The vibro-rammer seems to be preferred to any other type of apparatus for producing the results desired by M Fedi Such ramming apparently forces a settlement of the concrete onto the subgrade in such a way that no further settlement will be possible without erosion

I visited several jobs which M Fedi built, and examined them with great care The aggregate was all that could be desired A very lean mix had been used with a minimum water cement ratio, and the resulting concrete had been vibrorammed, steel brushed, and settled so thoroughly that during the ten year period that this pavement had been in use there was almost no cracking, little wear, and excellent service These pavements are in the heart of Paris and have been under heavy traffic for years

M Fedi believes that troubles with our pavements are too much cement, too poor an aggregate, and too little vibration To this he adds the necessity of making them skid-proof by removing the excess mortar from the surface

PLACEMENT OF CONCRETE BY VIBRATION ON FEDERAL AID PROJECT 425-E IN WISCONSIN

By Guy H Larson and William H Zamzow

Senior Assistant Highway Engineers, Wisconsin State Highway Commission

SYNOPSIS

In a field experiment conducted on a nine-mile paving project, the usual equipment found on the modern paving job was supplemented by a concrete vibrator of the Jackson tubular type, and by a mechanical longitudinal float The percentage of sand in the aggregate was reduced from that specified for ordinary placing methods in an effort to take advantage of the ability of the vibrator to handle harsher and drier mixes Test specimens consisted of cylinders and cores drilled from the pavement

The consistency and workability of the concrete on this job was controlled more by the floating and finishing operations than by the ability of the vibrator to compact concrete of a given consistency The use of the vibrator resulted in a 10 per cent increase in strength of the concrete on this project

During the season of 1937 the Wisconsin Highway Commission undertook a test project on which a Jackson Tubular Vibrator was used in laying 9 81 miles of concrete pavement The test was made in order to observe the vibrator and determine whether the beneficial effects of vibration demonstrated in the laboratory [1] could be obtained in a comparatively thin, flat section such as the slab on a paving project

The work was done on Federal Aid Project 425-E on United States Highway 16, between Columbus and Portage, in Columbia County The soil on the east end of the project was a sandy loam, grading to sand about the middle of the job, and to a sand-clay mixture at the west end The project was graded and the sand and sandy-loam sections, approximately six miles, were resurfaced with salvaged gravel and the entire project opened to traffic the season before the pavement was laid The section of sand-clay mixture compacted very well under traffic and was not resurfaced

The slab was of the standard thickened edge section for 20-ft pavement, 9 inches at the edges and tapering to a 6½-in uniform thickness 2 ft in from the edges The concrete was reinforced with No 4 gage cold-drawn steel wire fabric (6 by 6-in centers) placed 2 in beneath the surface of the finished pavement The area of the wires in the single layer of reinforcing used in the slab amounted to approximately 0 1 percent of the total end area of the slab

MATERIALS

Cement —The following are the average results of routine tests on the Marquette cement which was used on the project Fineness 92 6 percent, initial set 3 hr 15 min , final set 4 hr 50 min , tensile strength, 3 day 264 lb per sq in , 7 day 345 lb per sq in

Aggregates The aggregates were produced from a local deposit located near the project A sample of the pitrun material was washed, screened and tested, to determine the cement content and proportions to be used A portable screening and washing plant was then

[1] Journal of Research of the National Bureau of Standards, November, 1937, R P 1048

Journal of the American Concrete Institute, Reprint No 41, "Freezing and Thawing, Permeability and Strength Tests on Vibrated Concrete Cylinders of Low Cement Content" By M O Withey, Professor of Mechanics, University of Wisconsin

Proceedings of Sixteenth Annual Meeting, Highway Research Board, "Bond of Vibrated Concrete" By M O Withey, Professor of Mechanics, University of Wisconsin set up by the contractor to produce aggregates especially for this project Routine laboratory test data pertaining to the aggregates were as given in Table 1

The sand had approximately 1 5 percent silt by volume (average of field determinations 3 5 percent), and the colorimetric test showed it to contain no organic matter The mortar required 10 5 percent water for normal consistency, and developed tensile strength

TABLE 1

CHARACTERISTICS OF AGGREGATES

	Sand	Gravel
Wt lb per cu ft	117 0	109 9
Specific Gravity	2 69	2 69
Voids, per cent	30 4	34 7
Absorption, per cent	1 34	1 49

Sieve Analysis

Sand		Gravel	
Sieve number	Retained per cent	Sieve size in	Retained per cent
4	1	1½	0
8	25	¾	59
14	48	⅜	82
28	68	No 4	98
48	89		
100	98		
Fineness modulus	3 27		7 39

which was 171 percent of the standard at three days, 152 percent at seven days, and 145 percent at twenty-eight days

Water—The water was obtained from a small, spring-fed creek adjacent to the project Soundness, time of set, and strength tests on cement paste and mortar in which this water was used showed no change from those tests in which city tap water was used

EQUIPMENT

Mixer, Finishing Machine, and Joint Installer The mixer was a 1932 model, 27-E Rex, handling 33 cubic feet of concrete per batch It was equipped with automatic water-measuring and timing devices The mixing time was one minute The finishing machine was a 1930 model Lakewood with two screeds It carried the vibrator and accessories A Cleft-plane joint installer was used to install the premolded asphaltic ribbon used to form the longitudinal center joint of the slab This equipment was supplemented by the vibrator and a mechanical float

Mechanical Float: A power-driven mechanical float manufactured by the Koehring Company of Milwaukee was used for longitudinal floating of the surface This machine consisted of a heavy steel frame approximately 12 by 20 ft mounted on wheels traveling on the side forms and carrying a steel screed The screed was 1 ft wide and 10 ft long, with a 2-in turn-up at both ends The machine operated the screed back and forth longitudinally parallel to the center line of the pavement, and at the same time moved it diagonally across the pavement The overlap on the return trip was such that the surface was covered twice This float was experimental and is not a standard requirement in Wisconsin

Vibrator The vibrator was a Jackson Tubular Vibrator manufactured by the Electric Tamper & Equipment Company of Ludington, Michigan It consisted of a 4-in O D by 3/16 in wall, cold-drawn seamless steel (Shelby) tube, and a vibrating motor mounted slightly to one side of the middle of the tube A 4-in, 7¼-lb channel, held in place by steel clamps, was used to back up the tube The weight of the tube and channel member for a 19½-ft length was 293 lb In operation it was submerged in the mass of concrete in front of the screed to such a depth that the bottom of the tube was approximately 1½-in below the top of the finished pavement

The induction type vibrating motor was submersible and was designed with eccentric weights on its rotor shaft to produce vibration. It was attached directly to the vibrating tube by means of two steel clamps. The motor operated on a 110-volt, 3-phase, 60-cycle circuit and had a vibrating frequency variable from 3,600 to 4,800 r.p.m. It weighed 100 lb. A special-shaped filler plate weighing 60 lb., complete with clamps and bolts, was placed between the motor and tube. The weight of the complete vibrating assembly (tube, motor, filler plate, and clamps) was 483 lb.

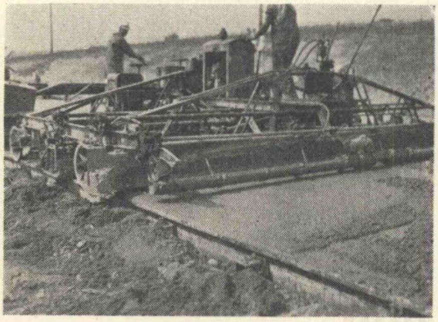


Figure 1. The vibrator in the raised position, showing the tube with vibrating motor mounted near the middle, and the mounting of the vibrating assembly ahead of the finishing machine screed. (The metal strips appearing as light-colored "posts" on the tube carry the needles used in making smoked-glass tracings of the movement of the vibrator.)

The vibrator mounting was of the hinged-arm type with cast steel frame and tube hanger brackets. From these brackets the tube was flexibly suspended on the finishing machine approximately 15 in. ahead of the front screed, with its ends about 3 in. from the forms. It was guided vertically in the horizontal position between the forms by means of heavy straps and rubber cushions or shock-absorbers. The brackets were adjustable to provide control of depth of submersion of the vibrator. The lifting ram was hydraulic, with a hand-operated pump mounted at the operator's position on the finishing machine and connected to the ram by means of a hose.

The power unit, mounted on the rear of the finishing machine was a 2,500-watt, self-excited, 60 to 80-cycle, 3-phase alternator, developing 110 to 140 volts. It was driven by a single cylinder, air-cooled, four to five horsepower gasoline engine, having a variable speed fly-ball governor.

Figure 1 shows a view of the vibrator in the raised position ahead of the finishing machine screed. Figure 2 shows the mass of concrete in front of the screed in which the vibrator was submerged and operated.

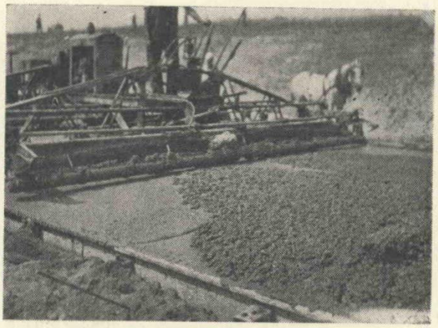


Figure 2. Mass of concrete in front of screed in which vibrator is submerged. Tops of metal "posts" visible above concrete.

SPECIAL PROVISIONS FOR VIBRATION

The State Highway Commission of Wisconsin "Standard Specifications for Road and Bridge Construction" were supplemented by the following "Special Provisions" covering the vibration of concrete on this project:

"*Concrete Pavement. Vibratory Finishing:* In addition to the requirements for striking off and consolidating the concrete pavement slab set forth in Subsection 401.13 (b) of the Standard Specifications, the concrete as soon as it has been placed shall be consolidated by means of a vibratory machine capable of transmitting the vibrations directly to the mass of concrete carried ahead of the front screed of the finishing machine. The vibrating element shall operate at not less than 3600 impulses per minute. The machine shall be of such design as not to produce excess mortar or laitance on the surface of the pavement, and shall not displace side forms, reinforcement or joints.

"It is desired to study the effect of vibration on the concrete and therefore the right is reserved to vary the proportions of the fine to coarse aggregates, by reducing the amount of fine aggregate, and proportionately increasing the amount of coarse aggregate, and also to reduce the amount of mixing water, to such a degree as will still produce workable concrete of proper consistency, with minimum amounts of fine aggregate and water, and maximum amounts of coarse aggregates; however the sum total amount of aggregate per sack of cement obtained by adding the amounts of fine and coarse aggregates, as set forth for the job mix for the particular source of aggregates will remain constant."

OPERATIONS

Preparation of Subgrade and Pouring Concrete: When paving operations were started the compacted surface of the grade was not disturbed. A cushion of fine sand of sufficient depth (approximately one inch) to clear high spots and level the surface was placed on the grade just ahead of fine grading operations. The subgrade was then shaped with the fine grader, compacted with a three-ton, self-propelled roller, and cut to true grade by means of a subgrade planer attached to the back of the mixer.

The concrete was placed in two layers; the first was struck off 2 in. below the top of the finished pavement. The steel fabric reinforcement was then placed, and the second layer of concrete was poured. When pouring the second layer the concrete batches were split and dumped in three longitudinal rows.

Vibrating and Finishing: The finishing machine made its first pass after the second layer of concrete was poured. During the first pass the vibrator was lowered and was operating in the mass of concrete just ahead of the screed as previously described. The second pass of the finishing machine was made with the vibrator raised and not operating. At the beginning of the work the vibrator was kept in operation right up to the transverse joints. It was found that this procedure pushed the joints over and, in spite of vibrating on the far side, left them in a tilted position. This difficulty was remedied by stopping the vibrator about 18 in. from the joint, raising it, and lowering it again just on the far side of the joint. The concrete generally presented a smooth, finished surface after

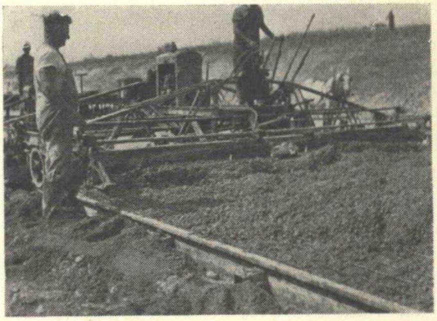


Figure 3. Finishing machine backing up after first pass. Note "dragging back" of concrete due to insufficient "left" of tube.

the first pass of the finishing machine. However, there appeared to be some "swell" or bulging of the concrete immediately back of the screed, due probably to pressure from the mass of concrete carried ahead and the setting of the screed. The screed picked up a considerable amount of concrete on the second pass of the finishing machine. Another factor contributing to this was that the "lift" of the vibrator was insufficient to completely clear the mass of concrete in which it operated so that when the finishing machine backed up there was a tendency for the vibrator tube to drag the concrete back a considerable distance on the surface, as shown in Figure 3.

The vibrator-finishing machine combination had no difficulty handling any of the consistencies and mixes used on the project. A few dry batches indicated that concrete with a considerably stiffer consistency could have been handled by this combination. Some honey-combing at the edges, however, indicated that the vibration was not as effective as it might have been at this point, and spading of the edges was necessary, particularly in the lower layer of concrete.

Placing of the asphaltic ribbon used to form the longitudinal plane of weakness or joint by means of the Cleft-plane joint installer resulted in some disturbance of the concrete at the middle of the slab, and was the cause of some trouble with the mechanical float which immediately followed the joint installer.

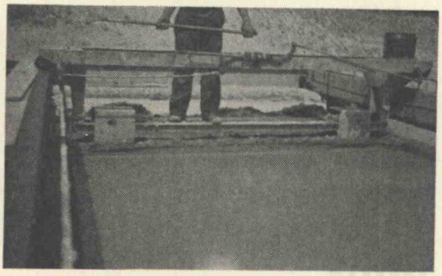


Figure 4. The mechanical float

Longitudinal floating of the pavement was done by the mechanical float described. Figure 4 shows the mechanical float in operation.

Some difficulty was experienced with the float cutting into and gouging the concrete. This tendency seemed to be aggravated by reductions in the percentage of sand in the aggregate, and by drier consistencies of the concrete. The trouble, however, was not due solely to the change in the mix since some difficulty was experienced with all mixes. The machine is a new development and improvements that are bound to be made should enable it to function very satisfactorily in any of the mixes used on the project. In this test, however, dryness and harshness of the concrete were controlled by the plasticity required for floating and finishing operations rather than by the inability of the vibrator to handle drier and harsher mixes. Straightedging, belting, brooming, and edging, all hand operations, followed the floating operation.

Curing: The concrete slab was cured by means of a wet burlap covering for the first 24 hours, followed by an impervious paper covering kept in place for 72 hours.

CONCRETE PROPORTIONS—SPECIMENS

Proportions: When the proportions for concrete for paving were designed for the particular materials used on this project, it was not contemplated that the concrete would be placed with the vibration method, and the mix specified was 1: 2.5: 4.3 by weight, which provided 37 percent sand in the aggregate. Some sections of pavement, using these proportions, were laid in the usual manner without vibration and some were laid with vibration. During the course of the test the percentage of sand in the aggregate was decreased by 2 percent intervals from the 37 percent originally specified to a minimum of 31.0 percent. Concrete having the reduced percentages of sand was placed by vibration. Changes in the proportion of fine to coarse aggregate were so made as to maintain the sum of the absolute volumes of these materials constant. Specifications limited the water to a maximum of 5.5 gal. per sack of cement, but the amount actually used was adjusted to the minimum that would produce the workability required by the various operations and conditions. Table 2 gives the mixes by weight and the proportions used per sack of cement.

The theoretical cement content of the concrete, using the above proportions with 95 percent of the maximum water, was 5 4 sacks per cubic yard of concrete An attempt was made to pour a section of 250 ft with each mix each day The order of pouring was changed so that a given mix would not be poured at the same time each day

Specimens Specimens for transverse strength tests consisted of 6 by 8 by 42-in beams cast in steel molds in accordance with the following procedure Immediately after the batch was dumped, concrete was shoveled from the batch into the mold The mold was filled slightly more than half full, and the concrete rodded 50 times with a ½-in rod The second layer, filling the mold to overflowing, was rodded 50 times and the edges of the specimen spaded with a 6-in sidewalk scraper The concrete was then struck off flush with the mold and finished with a steel trowel The specimens were covered with a piece of pavement curing paper The beams were placed alongside the slab upon being removed from the molds the following day, and were cured the same as the pavement until the seven-day tests were made Following the seven-day tests they were banked with sand with their upper surfaces exposed until subsequent tests Beams were made from the regular specified mix (No 1) only, because cylinders were also made from the concrete at the time of pouring, and it was planned to take cores from the pavement to determine the effects of vibration and make comparisons of compressive strengths and properties of the concrete from the various mixes as actually placed in the slab

The cylinders were cast in steel molds in accordance with A S T M standard procedure from the mixes poured each day They were cured the same as the pavement for four to five days, after which they were brought to the laboratory, placed outside and covered with damp sand until taken inside and prepared for testing Cores were drilled from the pavement when approximately two and one-half months of age They were taken to the laboratory and tested for absorption, density, specific weight, and strength

TABLE 2

CONCRETE PROPORTIONS

Mix No	Mixes (by weight)	Method of Placing	Proportions per sack of cement				
			Sand		Gravel (pounds)		Water
			Pounds	Per cent	¼ to ¾ in	¾ to 1½ in	Gallons per sack
5	1 2 51 4 32	Normal	236	37 0	162	244	Adjusted according to conditions Maximum of 5 5 gallons
1	1 2 51 4 32	Vibrated	236	37 0	162	244	
2	1 2 39 4 44	"	225	35 0	166	251	
3	1 2 26 4 57	"	212	33 0	172	258	
4	1 2 12 4 71	"	199	31 0	177	266	

TESTING

Transverse Tests The specimens were tested in a portable field-testing machine under center loading

Compressive Tests These specimens were tested in a 200,000-lb capacity Riehle testing machine with a free-moving head speed of 0 06 in per min Cylinders were removed from the sand and placed in water at room temperature for at least 18 hours before being capped at both ends with plaster of paris and tested Cores were capped at both ends with a 1 1 cement-sand mortar and tested in compression after the completion of absorption, unit weight, and density tests All cores and cylinders were in a saturated condition when tested

Absorption, Unit Weight, and Density Tests Upon being received at the laboratory the cores were examined for honey-comb and measured to determine their height, or the thickness of the pavement Then the lower end was trimmed preparatory to capping and the cores were allowed to dry in the laboratory air for two weeks They were then thoroughly brushed with a wire brush and weighed Following this the cores were allowed to soak in water at room temperature for 48 hours, after which they were re-weighed in air and in water The absorption in an air-dry condition and the weight per cubic foot of the concrete were computed from these data The density was obtained by dividing the unit weight thus determined by the computed unit weight of a solid mass of cement and aggregate in the proportions used on the job

TABLE 3

GENERATOR READINGS AND VIBRATOR FREQUENCY

Generator readings		Vibrator frequency R P M	Load on vibrator
Voltage	Amperes		
115	6 8	3500	Full
125	7 4	3600	"
105	3 4	3900	None

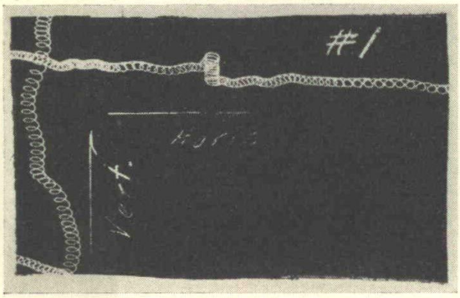
Frequency and Amplitude of Vibration The frequency of vibration of the vibrator was obtained by means of a "Frahm" hand-type, reed tachometer An attempt was made to obtain the amplitude of vibration by making smoked-glass tracings of the motion of the vibrator and taking measurements of these tracings Metal strips approximately 1 by 5¾ in were clamped in a vertical position at four different points on one-half the length of the vibrating tube It was assumed that vibrations of the tube would be symmetrical about its middle A phonograph needle was soldered in place in a hole in each of the metal strips 5½ in above its base These strips are visible in Figure 1, and Figure 3 Tracings of the motion of the submerged vibrator as it operated in the concrete were obtained by holding smoked-glass plates in contact with the needle points and moving them slowly in a vertical or horizontal direction Full-size pictures of a typical set of these tracings, taken simultaneously at the four points, are shown on Figure 5 A small section of each tracing, at a point where the motion of the glass was practically horizontal, or vertical, as the case might be, was enlarged ten diameters by means of photographic equipment Measurements were made of the enlarged tracings and the figures divided by ten to obtain the actual displacement The results are not considered to be accurate measurements of the amplitude of vibration, but it is felt that they afford a good indication of the comparative amplitudes and character of vibration at different points along the vibrator Different tracings indicated that amplitude and character of vibration varied with speed, and with the size and position of the mass of concrete on the tube The "combined amplitude" given beneath each plate is the square root of the sum of the squares of the vertical and horizontal amplitudes (one-half the displacement in each case)

Generator Readings· Simultaneous readings were taken of the voltage and current on the generator, and of the frequency of vibration of the vibrator under load and no-load conditions Table 3 shows a typical set of these readings

TEST RESULTS

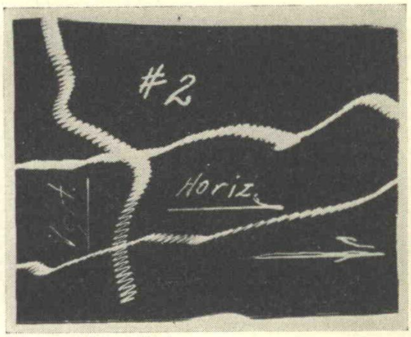
Comparison of Physical Properties. A general summary of individual test data, grouped according to mixes, is given in Table 4.

Referring to Table 4, mixes Nos. 1 and 5 had the proportions specified for normal placing of the concrete; mix No. 5 was poured without vibration, while mix No. 1 was vibrated. Mixes Nos. 2, 3, and 4 had successive reductions in the percentage of sand and were all vibrated. There were practically no differences in the water-cement ratios and slump of these mixes, the only material variation being a tendency for mix No. 4 (31 percent sand) to slump less than the others.



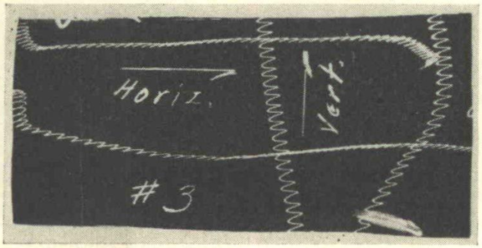
10 feet. 2 inches from right end

Vertical Amplitude .... 0.023 in.
Horizontal Amplitude .... 0.037 in.
Combined Amplitude .... 0.039 in.



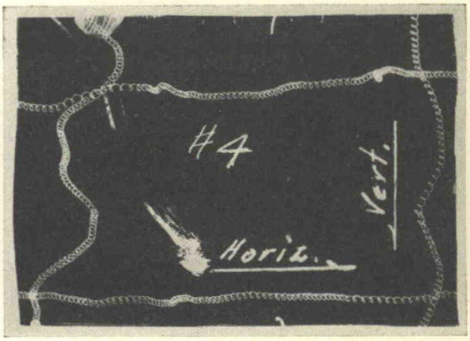
6 feet, 2 inches from right end

Vertical Amplitude .... 0.017 in.
Horizontal Amplitude .... 0.040 in.
Combined Amplitude .... 0.044 in.



2 feet, 2 inches from right end

Vertical Amplitude .... 0.008 in.
Horizontal Amplitude .... 0.038 in.
Combined Amplitude .... 0.039 in.



at right end

Vertical Amplitude .... 0.017 in.
Horizontal Amplitude .... 0.018 in.
Combined Amplitude .... 0.025 in.

Note: Pictures show actual size of tracings. "Right end" in notations above refers to the right end of the vibrator when facing the same direction as the finishing machine. Arrows indicate the horizontal and vertical directions with respect to the tracings.

Figure 5. Smoked-glass tracings of motion of vibrating tube. (Taken simultaneously at four different points on the tube.)

Effect of Reductions in the Percentage of Sand: Reductions in the percentage

TABLE 4
SUMMARY OF DATA

	Cement[1] content		Water[2]		Slump in	Strength—lb per sq in									Core Tests[4]		
						Beams			Cylinders			Cores					
						Age—Days			Age—Days			Position[3]					
	A	B	G	W/C		7	14	28	7	28	97	E	M	C	Sp weight lb	Density per cent	Absorption per cent
Mix No 1—Vibrated—36 8 Per cent Sand																	
No of Spec	19	19	19	19	18	14	1	15	17	17	17	12	12	12	12	12	12
Maximum	5 52	5 45	5 41	0 72	1 50	740		817	4500	5020	5680	7150	7560	7300	158 2	92 5	1 552
Minimum	5 36	5 37	4 68	0 62	0 75	540		635	2430	2950	3830	5550	5190	5520	155 5	90 8	0 870
Average	5 40	5 42	5 07	0 67	1 08	625	710	726	3270	4075	4781	6443	6318	6222	156 4	91 5	1 179
Mix No 2—Vibrated—35 0 Per cent Sand																	
No of Spec	16	16	16	16	16				10	10	10	13	13	13	13	13	13
Maximum	5 44	5 57	5 32	0 71	1 00				4110	5050	6160	6460	6960	7100	157 5	92 1	1 536
Minimum	5 36	5 39	4 09	0 55	0 75				2660	3250	3400	5150	3030	4920	155 1	90 7	0 955
Average	5 39	5 45	4 89	0 65	0 97				3223	4026	4774	6026	5802	5958	156 3	91 4	1 199
Mix No 3—Vibrated—33 Per cent Sand																	
No of Spec	12	12	12	12	11				6	6	6	10	10	10	10	10	10
Maximum	5 44	5 57	5 30	0 70	2 25				3800	4750	5330	6720	7400	7070	157 2	92 0	1 646
Minimum	5 36	5 39	4 09	0 64	0 05				2688	3170	4110	4620	4830	4520	154 4	90 3	0 981
Average	5 39	5 44	4 93	0 65	1 05				3059	3643	4380	5691	5917	6042	156 2	91 4	1 233
Mix No 4—Vibrated—31 Per cent Sand																	
No of Spec	7	7	7	7	7				5	5	5	7	7	7	7	7	7
Maximum	5 44	5 54	5 26	0 70	1 00				3763	5240	5340	6220	6510	6770	157 8	92 3	1 424
Minimum	5 36	5 39	4 30	0 57	0 00				2240	2610	3430	5270	4660	5190	155 9	91 2	0 916
Average	5 39	5 45	4 85	0 65	0 61				3035	3838	4344	5897	5719	5930	156 7	91 7	1 202
Mix No 5—Not Vibrated—36 8 Per cent Sand																	
No of Spec	8	8	8	8	8							8	8	8			
Maximum	5 44	5 48	5 26	0 70	1 5				Same As			6350	6720	6640	155 8	91 1	1 790
Minimum	5 36	5 39	4 68	0 62	0 5				Mix No 1			4950	4730	3830	154 1	90 1	1 030
Average	5 39	5 42	5 06	0 67	1 06				3270	4075	4781	5774	5913	5505	155 2	90 7	1 369

1—A is as computed from cement used and area of pavement
B is as computed from the absolute volumes of the materials, including water
2—G = gallons per sack of cement, W/C = water-cement ratio by volume
3—E = 8 ft 3 in from center line of pavement, M = 5 ft from center line, C = 1 ft 3 in from center line
4—Averages for three cores listed under E, M, C

of sand did not permit any consistent reduction in the amount of water. The cement content computed from the absolute volumes of the materials varied slightly with changes in the amount of water, but had a close relationship with the cement content as computed from the amount of cement used and area of pavement laid. The compressive strength as indicated by the cylinders apparently dropped as the percentage of sand

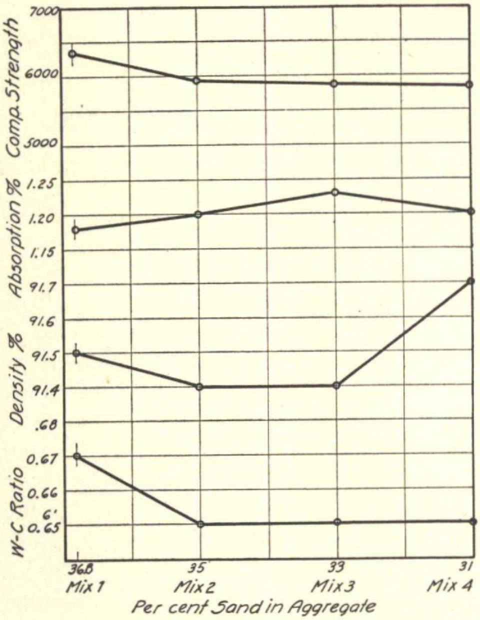


Figure 6. Relation between percentage of sand, water-cement ratio, density, absorption and strength of concrete in pavements.

dropped. Referring again to Table 4, which shows the results of tests on concrete in the pavement as represented by the cores, there is no appreciable increase in specific weight and density, or change in absorption with reductions in the percentage of sand. The compressive strengths of the cores taken from the pavement exhibit the same tendency as was indicated by the strengths of field cylinders; that is, a slight decrease in strength with reductions in the percentage of sand.

Figure 6, showing "Relation Between Percentage of Sand and Density, Water-cement Ratio, Absorption, and Strength of Concrete in Pavement," is a graphical presentation of the data obtained from the cores. Figure 7 shows a representative core from each mix. There was little apparent difference between these cores except that the bottoms appeared to become slightly rougher as the sand was

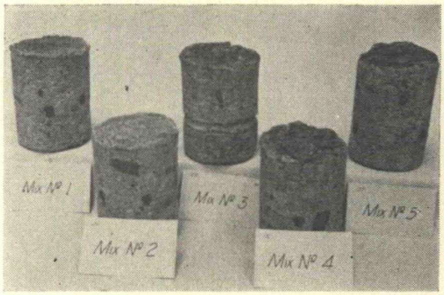


Figure 7. Representative cores from the different mixes

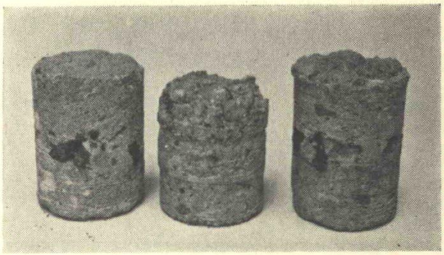


Figure 8. Cores showing honey-comb, mix number 5

reduced. Mix No. 4, with 31 percent of sand but which was vibrated when placed, showed fully as good a bottom as mix No. 5 with 37 percent of sand, but which was not vibrated when placed.

Figure 8 shows the only cores having honey-comb. The end cores were honey-combed at the steel and the middle core had a slightly honey-combed bottom. These cores all came from mix No. 5, non-vibrated concrete having 37 percent sand.

Effect of Vibration. Table 5 shows the results of tests on cores taken from vibrated and non-vibrated concrete using the same proportions. Vibration resulted in a very slight increase in specific weight and density, a slight decrease in absorption, and an increase of 10.5 percent in strength.

TABLE 5

EFFECT OF VIBRATION ON CONCRETE AS INDICATED BY RESULTS OF TESTS ON CORES

Mix Number	W/C by volume	Unit weight (lb per cu ft)	Density (per cent)	Absorption (per cent)	Compressive strength (lb per sq in)
1 Vibrated	0.67	156.4	91.5	1.18	6330
5 Non-vibrated	0.67	155.2	90.7	1.37	5730
Ratio $\frac{\text{Vibrated}}{\text{Non-Vibrated}}$	1.00	1.008	1.008	0.86	1.105

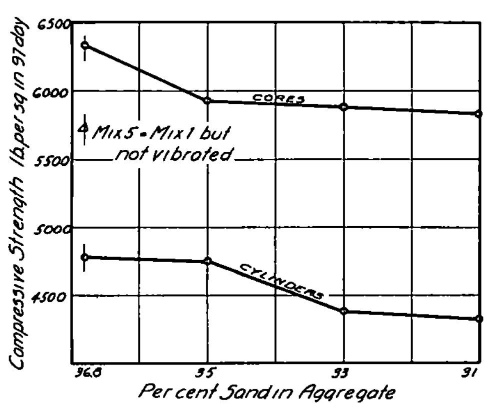


Figure 9. Relation between compressive strengths of cores and cylinders at same age

Strength of Concrete as Indicated by Test Cylinders and as Indicated by the Cores. Figure 9 shows the average results of compressive strength tests on cores and cylinders from the various mixes. Results on the cores indicate that the strength of the concrete in the pavement is from 24 to 35 percent greater for the vibrated concrete, and 20 percent greater for non-vibrated concrete than that indicated by the test cylinders made at the time the concrete was poured.

Strength of Concrete at Different Points in the Slab. It was noted that the vibrating tube did not vibrate exactly the same throughout its length. Cores were taken at different distances from the center line of the pavement to ascertain the effect of this difference in vibration on the concrete. The results of strength tests on cores taken near the center line, at the middle of the 10-foot slab, and near the edge, are shown in Table 4. The figures indicate that there was no consistent difference in the strength of the concrete at these points.

Comparison of Concrete on Vibrated and Non-Vibrated Sides of Joints. As stated under "Vibrating and Finishing," it was impracticable to continue vibration right up to the joint as the finishing machine approached it. Table 6 shows comparative test results obtained on cores taken from the vibrated and non-vibrated sides of the joint. The results indicate there were no differences in the strengths, densities, or absorption of the concrete on the two sides of the joint. Cores taken directly over the joint showed the concrete to be dense and well compacted adjacent to it.

SUMMARY

In concluding, we wish to repeat that the consistency and workability of the

TABLE 6

SUMMARY OF TESTS ON CORES TAKEN ADJACENT TO JOINTS

Vibrated side of joint				Non-vibrated side of joint			
Core Number	Strength	Density	Absorption	Core Number	Strength	Density	Absorption
157	6,280	90 9	1 3	156	6,230	90 8	1 3
160	7,260	91 6	1 1	159	7,160	91 9	1 0
162	5,040	91 6	1 3	161	5,470	91 0	1 5
164	6,000	92 0	1 4	163	5,840	90 2	1 4
Average	6,150	91 5	1 3	Average	6,180	91 0	1 3

concrete on this project appeared to be controlled by the plasticity required in the operations of floating and finishing subsequent to vibration, rather than by the ability of the vibrator to compact concrete of a given consistency The results obtained under the conditions prevailing may be summarized as follows

1 Use of the vibrator in placing the concrete resulted in an increase in strength of approximately 10 per-cent
2 Reductions in the percentage of sand in the aggregate were not effective in increasing strength, probably because other factors prevented reductions in the amount of water
3 Variations in vibration of the tube at different points did not seem to be reflected in the strength of the concrete
4 Vibration seemed to be effective on both sides of the joint even though the vibrator did not operate right up to the joint on the approach side
5 The concrete in the pavement was considerably stronger than was indicated by compressive test specimens cast at the time the concrete was poured, and cured outside in damp sand

It is intended to study the behavior of the slab under traffic and climatic conditions in order to determine if possible what effect vibration may have on the service of the pavement

EXPERIENCE WITH CONCRETE PAVEMENT VIBRATORS IN KANSAS

By R D Finney

Engineer of Materials, Kansas Highway Department

SYNOPSIS

Three types of vibrators were used on the different projects studied in this report A front screed vibrator was used on one project, a vibrating pan on another, and the vibrating tube on five projects In addition to study of the projects upon which vibration was used, nearby projects using the same aggregates and cements were studied in an effort to determine the effects produced by vibration

A fair degree of satisfaction has been obtained by using vibrators in this State Although gains in slab quality have not been as great as it is possible to obtain, only experience can bring about high efficiency The statement that the higher the frequency, with proper amplitude, the better the results, is found true in Kansas as in other places It is believed also that the more internally a vibrator may be made to work, the more favorable the result

Vibration as a means of handling and placing concrete has had a limited use in Kansas for several years In the Highway Research Board Proceedings for 1935, Mr R B Wills reported upon some of the earlier experiences The 1937 construction season has seen a very definite swing towards vibration in Kansas concrete paving practice The specifications for concrete pavement construction in Kansas are based upon a maximum water content of 5 75 gallons per bag of cement, and a minimum cement content of 1 25 barrels per cubic yard of concrete This specification applies where the aggregate consists of a suitable combination of fine and coarse aggregate With such mixtures the use of a vibrator is optional with the contractor In constructing pavements in which the quantity of coarse aggregate is insufficient, the use of a vibrator is not permitted

This year's increase in the use of vibration has come from realization by the contractors that it is not possible to reduce the cement factor to a figure approaching 1 25 bbl per cu yd without vibration, and the added fact that the recently developed tube type of vibrator is low in cost and readily adapted to the existing handling and placing machinery

Three types of vibrators were used on the projects studied in this report A front screed vibrator was used on one project, a vibrating pan on one, and the vibrating tube on five Nearby projects using the same aggregates and cements but without vibration were also studied in an effort to determine the effects of vibration

Design of Concrete Mixtures. No set method for the design of the concrete mixture is used on Kansas projects Essentially the trial mix method is used, and in some cases preliminary figures based upon the mortar voids theory and Abrams Fineness Modulus theory are developed, depending upon the experience of the engineer in charge Complete data as to proportions used, and the weight of the resulting concrete were always available, and from this the constants shown in Table II were computed

It is believed these figures show that very satisfactory results were obtained in the design of the mixes used

COMPARISON OF JOB RESULTS BETWEEN REGULAR AND VIBRATORY PLACEMENT

Group 1 In Group 1, job 1 was laid with an Ord finishing machine by the same contractor as job 1A, in which he used a tubular vibrator attached to the same finishing machine The aggregates, crushed limestone for coarse aggregate and river gravel for fine aggregate, were from identical sources and had almost identical gradings. Job 1 consisted of 7.12 miles of single slab and job 1A of 11.0 miles of single slab. The vibrator was able satisfactorily to compact a mix using a slump of $1\frac{3}{8}$ in., where the regular finishing machine needed concrete with a $2\frac{1}{4}$ in. slump. About $2\frac{1}{2}$ per cent less sand, by volume, was in the mix compacted by the vibrator with about the same ease of finishing. The differences in results secured by the vibrating and non-vibrating processes are shown by the following figures:

Specific gravity of cores, vibrated 2.33, tamped 2.29.

Weight of green concrete, lb. per cu. ft., vibrated 151.5, tamped 149.5.

Cement factor, bbl. per cu. yd., vibrated 1.26, tamped 1.33.

Compressive strength, lb. per sq. in., vibrated 5426, tamped 5022.

On this job the tube was attached to an old type Ord Finishing Machine, which was lighter than the present day model. At the very beginning it was evident that the finishing machine would not have enough traction to move the tube forward through the mass of concrete. Weighting the machine with railroad rails did not prevent slippage of the wheels and caused the forms to settle. The contractor then devised a four wheel drive for the machine with only a little added weight which enabled the machine to go forward more rapidly, though slippage was not entirely eliminated. Only one pass was made with the vibrating tube. The second time over only the screeds were used.

A mix was designed for this vibrator job for a total Fineness Modulus of 6.15, an increase of 0.20 over the Fineness Modulus of a calculated normal mix using those aggregates. By normal mix is meant one calculated for a job to be finished with ordinary methods. It is interesting to note that in designing a concrete mix for use with the vibrator an increase in b/bo of 0.06 over the ratio of a normal mix was necessary to give a satisfactory sand content.

Group 2: In Group 2 on jobs 2, 2A and 2B a regular finishing machine, a vibrator pan and a tubular vibrator were used respectively. Cement factors were about the same, 1.33. Very little difference was given by the vibrator.

The aggregate on job 2B contained 28 per cent of sand by volume, which is obviously low. The coarse aggregate was obtained by crushing a very highly cemented sandstone. The fine portion of this aggregate (minus 4 mesh) consisted mainly of fine sand, therefore, the indicated sand percentage is rather misleading. The fine aggregate was river sand. That in turn explains why the ratio of b/bo of 0.85 could be used. On most of these jobs the ratio b/bo was raised by 0.06 above that necessary to obtain a satisfactory job using a regular finishing machine. In this case it was raised 0.10, which, if the fine and coarse aggregate were combined and separated again, would produce a ratio of b/bo of 0.81.

Figure 1. Skip Discharging Ready for Vibration

On this job, as in Group 1, the use of the tubular vibrator presented a problem since it was placed on a very poor finishing machine. This machine was not quite as light as the one used on job 1A, but it had seen too many years service to have sufficient traction to move the tube through the concrete in a satisfactory manner. Possibly another factor entering into this was the fact that the fines in the coarse aggregate were somewhat of a quicksand variety, which did not give a very workable mix even though the percentage was low. Test data on cores are not yet available.

Group 3: Job 3 was placed with a regular finishing machine with tamper. Jobs 3A and 3B were placed with the front screed vibrator. The aggregates were of similar grading in all three cases. The coarse aggregate was crushed limestone and river gravel was the fine aggregate. This vibrator allowed about the same increase in Fineness Modulus and ratio

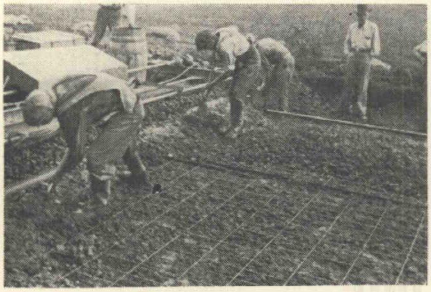


Figure 2. Vibrator in Action

of b/bo over the mix used in the regular finishing machine as did the tubular vibrator. The Fineness Modulus was 6.20 and the b/bo ratio was 0.87. The tabulated data presents most of the information of interest on this job. The cores taken from the pavement recently show it to be in excellent condition as respects honey-comb, but as yet the specific gravity results are not available.

Group 4: In Group 4, job 4 was placed and finished with a Lakewood finishing machine, and job 4A with a tubular vibrator attached to an ordinary finishing machine. It is interesting to note the effect of using a coarse aggregate consisting of rounded hill gravel. Ordinarily coarse aggregate in Kansas consists of crushed limestone with a specific gravity of about 2.62. The specific gravity of this particular hill gravel coarse aggregate was 2.42. As shown in Table I it was not deemed good practice to reduce the sand content below that used in concrete for placement and finish by an ordinary finishing machine. However, the slump was reduced from 2 in. using the ordinary finishing machine to $\frac{7}{8}$ in. using the vibrator. That reduction in slump did not do much toward reducing the cement factor, due to the fact that less coarse aggregate was used in the vibrated mix than in the mix with the regular finishing machine. Compressive strengths were increased from 5315 to 5840 lb. per sq. in. by the vibrator. Average core specific gravities of 2.23 are considered good for these aggregates.

Group 5: A comparison of these two jobs, 5 using the regular finishing machine and 5A using the tubular vibrator, merely serves to support the information shown above, mainly that an increase of 0.2 in the Fineness Modulus and of 0.06 in the ratio of b/bo seems to be about right in converting from a normal to a vibrated mix.

As on other jobs the finishing machine did not have enough traction to push the tube vibrator through the mass without slippage. A reduction in cement factor from 1.30 to 1.265 was possible through the use of the vibrator. In terms of money saved in cement costs, this amounts to about $250 per mile, with other possible economies in finishing operations, due to the fact that the finished slab after the vibrator has been across it does not have "hard" and "soft" spots, thus producing a better riding surface with less work. It is not unlikely that the elimination of these frequent "hard" and "soft" spots in the green concrete will subsequently provide a better riding surface.

Group 6: Both of these jobs, 6 and 6A, were placed with the tubular vibrator. Due to the fact that there are no jobs built in the vicinity of these chat aggre-

TABLE I
CHARACTERISTICS OF AGGREGATES

Group		Length	Sp gr of		Dry and rodded wt		Ab vol of		Voids		Sieve analysis															
											Fine aggregate percentage retained								Coarse aggregate percentage retained							
No	Sec		Fine	Coarse	Fine	Coarse	Fine	Coarse	Fine	Coarse	⅜	4	8	16	30	50	100	G F *	2″	1½″	1″	¾″	⅜″	4	8	16
1	1	9 566	2 62	2 62	111	102	6789	6239	3211	3761	0	1	10	30	58	93	100	2 92	0		45		88	97		
	1A	5 50	2 62	2 62	111	100	6789	6117	3211	3883	0	2	10	32	56	93	100	2 93		20	52	66	87	99		
2	2	6 262	2 63	2 62	107	100	652	612	348	388	0	0	0	12	58	94	100	2 64	0		43		88	96		
	2A	2 456	2 64	2 62	109	100	6616	6117	3384	3883			1	28	68	94	100	2 91	1		52		83	97		
	2B	2 279	2 61	2 63	113	110	6938	6093	3062	3907	0	2	13	32	55	82	99	2 84	0	25	52	66	87	96	98	98
3	3	607	2 63	2 64	111	95	68	575			0	1	7	27	60	94	100	2 89		0		55	83	97		
	3A	3 423	2 63	2 64	111	95	67	58			0	1	7	23	52	95	99	2 77	0		52		89	97		
	3B	3 287	2 62	2 60	109	93	6667	5732	3333	4268	0	1	8	21	53	93	100	2 76	4	27	50	64	85	98		
4	4	6 725	2 62	2 42	112	93	685	614	315	386	0	1	13	30	56	87	98	2 85		5		56	85	97		
	4A	10 59	2 62	2 42	112	93	6851	6160	3149	3840	0	1	14	38	66	92	99	3 10		4		53	84	97		
5	5	1 987	2 62	2 63	110	94 5	6728	5759	3272	4241	0	0	7	27	59	93	99	2 88	0		47		85	97		
	5A	2 240	2 63	2 64	110	95	6703	5769	3297	4231	0	2	8	29	59	93	99	2 88	0		49		86	98		
6	6	4 344	2 62	2 61	103	93	63	57	37	43	0	0	2	23	59	81	91	2 56					13	69	92	97
	6A	3 869	2 60	2 50	95	85	586	545	414	455	0	0	2	22	54	78	89	2 45					5	67	96	99

* Graduation Factor or Fineness Modulus

gates since Kansas has been on the $5\frac{3}{4}$ gallons of water per sack of cement, no comparisons of a normal chat mix placed with the regular finishing machine with tamper and those placed with the vibrator are available. However, considerable interest was shown in these particular jobs because from a theoretical combination of coarse and fine chat aggregates, the cement factor would approximate 1.80 bbl. per cu. yd. The use of the vibrator, however, permitted slumps of between $\frac{1}{4}$ and $\frac{3}{8}$ in., which in turn lowered the cement factor to 1.58, indicating a considerable saving in the cost of cement. Chats consist of very hard cherty stone having exceedingly sharp fractures, and the grading of this by-product of zinc mining is very poor (Table I). Under normal conditions it takes an excessive amount of mortar paste to provide lubrication enough for such a combination of aggregates to be in a workable condition without the use of some external means of compaction. Apparently the tubular vibrator gives the necessary aid to allow the sharp particles to slip readily by one another into place.

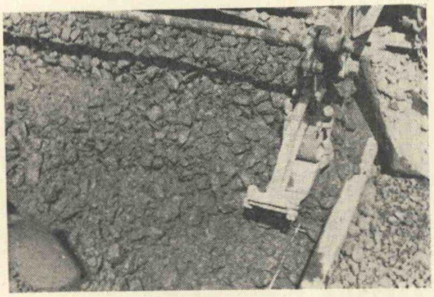


Figure 3. Closeup of Action

Figure 4. Vibrator Withdrawn

Another point of interest is the exceedingly low apparent specific gravity of the cores taken from these pavements. They are quite indicative of the porous structure of the slab, but it will be noted that the compressive strengths were quite high.

A design mix for chat aggregates is difficult to work out due to their peculiar characteristics and to the fact that various sources do not produce similarly graded aggregates of similar specific gravity. A comparison of these two projects, each of which obtained their aggregate from different sources, will demonstrate the variety of gradings and specific gravities that are obtained from these chat fields.

GENERAL COMPARISONS

(1)—A study of the summarized results indicates that at least satisfactory quality of pavement concrete is produced by either method of compaction and finishing. Several projects have shown results obtained by vibration to be superior. Generally, the apparent specific gravity of cores taken from the pavement have been slightly higher for vibrated concrete. The average 90 day compressive strengths have also been greater for vibrated pavements. This may or may not be attributed to vibration.

From the contractors standpoint there may be a slight advantage in finishing costs and cement costs.

(2)—One important factor which should be brought out in this discussion is the rigid requirement that the concrete as it is placed between the forms must be in its final position before any tube vibrator or other finishing machine passes over it On practically all vibrated paving jobs the first point of observation was the fact that the puddlers, or the laborers in front of the finishing machine, had the impression that the purpose of passing the vibrator over the concrete was to move the mass deposited by the mixer into the voids that had been left in placing the concrete between the forms This without question is one of the practices that must be overcome before placing concrete by vibration will be highly successful The purpose of the vibrator is not to level off the mass of concrete that is deposited on the grade between the forms, but to pack the aggregates that have been placed in their final position by the mixer and the puddlers combined, into the closest possible space

Generally speaking, and particularly with a vibrated concrete mix, the edges of any concrete mass consist mainly of coarse aggregate Obviously, when the percentage of excess mortar is reduced for proper vibration, there will not be sufficient mortar carried forward by the vibrating unit to fill in the "nests" of coarse aggregate occurring at the base of each adjacent pile For this reason it is essential that the concrete mass be in a condition that only compaction is required for final placement

The foregoing discussion is simply a part of the educational program that must be instituted if we are to receive the utmost in benefits from vibration of concrete As soon as the engineer on the job begins to understand the real purpose of vibration and how the vibrator can be made to act most efficiently, then will we be getting the true benefits that accompany slab vibration

(3)—*Concrete Mix Design* Very satisfactory results have been obtained on certain jobs by designing normal concrete mixes by using the Abrams Fineness Modulus theory Briefly, that is, for any maximum size aggregate, there is quite definitely a given Fineness Modulus that should be used which will give a balanced proportion of rock and sand Thus, regardless of the grading of any ordinary individual aggregate, fine or coarse, a workable mix may still be produced, which is neither harsh nor fat by using the total Fineness Modulus indicated for that maximum size. In adapting this method to vibrated mixes it is simply a matter of raising the Fineness Modulus over that which would be used for a normal mix Accumulated information indicates the increment should be about 0 2 While this method of design is not perfect, if used correctly, it is workable with only minor adjustments

Regarding the use of the Talbot Richards formula, particularly the b/bo or workability factor, more study must be made However, the values as shown in Table II indicate that for each workable mix there are probably variable b/bo values depending upon the individual aggregates, but as soon as the proper b/bo value is determined for a normal mix, increasing that value produces a ratio of rock to sand that is quite close to that desired for the vibration process

(4)—*Types of Vibrators* Mixes and results obtained from three types of vibrators have been given Comments as to their relative efficiency cannot be made until more information concerning their use and results is secured However, regardless of the type of vibrator used, the finishing machine to which it is attached must be in excellent structural condition, have reserve power and have good traction This statement is probably more true for the tubular vibrator than for other types because of the fact that a tubular vibrator may be extended into the mass of the concrete to a depth at which it takes great force to move it forward through the mass Regardless of the efficiency of the vibrator itself,

TABLE II

CONCRETE

Group No	Group Sec	Mix	Slump in	Wt per cu ft green concrete	Cement factors Yield	Cement factors Actual	Cement factors Theor	Ave 90 days compressive	Specific gravity of cores	Type of slab compaction	Sand by vol %	Total fineness modulus	a/c	b/bo	Theor sp ave M/V	Theor sp ave Gr green concrete	Theor sp ave Sp gr cores
1																	
	1	1 2 06 3 78	2¼	149 5	1 33	1 34	1 34	5022	2 29	Regular finishing machine	35 3	5 95	2 62	75	1 87	2 44	2 29
	1A	1 2 11 4 30	1⅜	151 5	1 262	1 31	1 27	5426	2 34	Jackson Tube Vib (Gas-Elec) on regular finishing machine	33 0	6 14	2 91	81	1 61	2 45	2 33
2																	
	2	1 2 16 3 84	2	150 0	1 34	1 35	1 34	5974		Regular finishing machine	36 0	5 88	2 87	76	1 79	2 42	
	2A	1 2 14 4 16	1½	151 5	1 325	1 355	1 29	5932	2 35	Vibrating pan between screeds of regular finishing machine	34 0	6 04	2 88	81	1 65	2 48	2 35
	2B	1 1 67 4 29	1¼	151 5	1 30	1 343	1 30			Jackson Tube Vib (Gas-Elec) preceding regular finishing machine	28 0	6 04	2 37	85	1 44	2 44	
3																	
	3	1 2 02 3 88	2	150 5	1 38	1 45	1 37	6064		Regular finishing machine	34 0	5 97	2 79	80	1 59	2 44	
	3A	{1 2 28 4 36	1¼	150 0	1 252	1 29	1 27 }	6148		Front screed electric vibrator	{34 0	6 04	3 12	82	1 53}	2 41	
		{1 2 39 4 24	1	150 0	1 253	1 33	1 266}				{36 0	5 94	3 27	80	1 64}		
	3B	1 2 06 4 60	1½	148 0	1 26	1 28	1 264			Front screed electric vibrator	31 0	6 20	2 79	87	1 35	2 43	
4																	
	4	1 2 26 4 21	2	144 5	1 252	1 282	1 25	5315		Regular finishing machine	35 0	5 83	3 16	78	1 73	2 34	
	4A	1 2 19 4 06	⅞	144 0	1 27	1 29	1 28	5840		Jackson Tube vibrator preceding regular finishing machine	35 0	5 88	3 06	77	1 73	2 33	2 23
5																	
	5	1 2 28 4 05	2¼	150 5	1 30	1 345	1 31	5755	2 33	Regular finishing machine	36 0	5 94	3 12	79	1 58	2 44	2 34
	5A	1 2 16 4 54	1¼	152 0	1 265	1 31	1 26		2 30	Jackson tube vib (Gas-Elec) preceding regular finishing machine	32 0	6 17	2 96	85	1 42	2 44	2 31
6																	
	6	1 2 10 2 80	⅜	142 0	1 56	1 58		6255	2 17	Jackson tube vib (Gas-Elec) preceding regular finishing machine	40 0		2 69	67	2 14	2 38	2 17
	6A	1 2 10 2 80	¼	137 0	1 59	1 65	1 68	5053	2 07	Jackson tube vib (Gas-Elec) preceding regular finishing machine	40 0		2 51	70	1 94	2 33	2 07

* Average of 30 or more beams

the benefit derived from vibration is destroyed, or lessened considerably, if the finishing machine is not in good condition

The subject of frequency of vibrators may be dismissed with the statement, which is in agreement with those who have used vibrators to any extent, that the higher the frequency, with proper amplitude, the better the results that will be obtained Sufficient data are not at hand to indicate proper values for frequency and amplitude

Results from using vibrators in Kansas have been fairly satisfactory While probably not as much has been gained in slab quality as may be possible through the use of vibration, it is certain that nothing has been lost through their use Only through experience will high efficiency be obtained from them

The Kansas Highway Department is insisting at the present time, however, that if a vibrator is to be used for pavement concrete, the finishing machine must be up to standard, and the engineer on the job must insist upon careful placement of the concrete The concrete must be in its final position between the forms prior to vibration The vibrator is to be used only as a means of settling the mass and not as a means of spreading

It is believed that of the several types of vibrators used on this work, the more internally a vibrator may be made to work, the better the results will be

INVESTIGATION OF VIBRATORY METHOD OF FINISHING CONCRETE PAVEMENT IN ILLINOIS

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SYNOPSIS

This investigation was conducted to study the practicability of the vibratory method of placing concrete on a regular paving contract Mixtures of standard proportions placed by conventional methods were compared with two types of mixtures placed by vibration, in which the proportion of fine to coarse aggregate was adjusted to give the best results Both gravel and crushed stone coarse aggregates were used In one type mixture, designed for standard yield, a reduction in mixing water of about one-half gallon per bag of cement and a corresponding increase in strength were obtained In the other, in which the standard water-cement ratio was maintained, a saving in cement of about 10 per cent was obtained without reduction in strength The progress of the work was as satisfactory as with the conventional methods and the surface of the pavement showed no defects after one year The relative amounts of honeycomb in the slabs showed that the vibratory method is far superior to the Illinois Standard method in consolidating the concrete Concrete of one inch slump, which was about as dry as could be discharged and spread by the mixer bucket, was consolidated very satisfactorily around transverse metal joints, when installation bars to protect the copper seal or bituminous cap were used

This investigation takes up the practical application of the vibration method under field conditions Before presenting the data, however, it will be of advantage to consider briefly the results of a previous investigation in Illinois, because the investigation described herein may be considered, in a way, a continuation of the first one

INVESTIGATION OF 1932

In August, 1932, the Illinois Division of Highways constructed about one mile of pavement with one of the first vibratory finishing machines offered for use, with the object of making a thorough study of the merits of this method of placing and finishing concrete pavement Only a very brief account of this investigation can be given here, and for further information, a report entitled, "Vibratory Method of Finishing Concrete Pavements," issued November 15, 1933, should be consulted

Description of Finishing Machine The vibratory finishing machine used in the investigation of 1932 was essentially the standard double-screed equipment with screeds slightly different in design and provided with electrically driven vibrator units operating at about 3,600 impulses per minute Four vibrator units were placed on the front screed and two on the rear screed and the vibratory action was imparted to the concrete through the screeds while in motion

Mixtures Used The experimental pavement contained 114 individual sections, averaging about 45 ft in length, three different series of progressively changing concrete mixtures for both gravel and crushed stone coarse aggregate were used Expressing all quantities in amounts per bag of cement, the mixing water was the major variable in one series, the coarse aggregate in another, and the fine aggregate in another An idea of the range of mixtures used may be had from the fact that the quantity of cement per cubic yard of concrete ranged from 1 11 barrels to 1 58 barrels A sufficient number of mixtures were used to determine the maximum practical limits of the proportions of materials which could be handled by this machine and the proportions which appeared to be most suitable

The ability of the vibratory finishing machine to handle and consolidate even the harshest mixtures was found to be truly remarkable and satisfactory pavement was obtained from most, if not all, of the mixtures placed The most suitable mixtures, however, appeared to be those having a slump of about one-half inch which contained a ratio of sand to the total quantity of aggregates by absolute volumes in the vicinity of 32 per cent for gravel mixtures and 35 per cent for crushed stone mixtures

Surface Condition The only disconcerting thing about the investigation was that some surface scaling developed on many of the sections However, scaling occurred on several other pavements constructed that year and therefore it could not be said that the scaling which occurred on the experimental pavement was due to the vibratory method of compacting and finishing, although an excessive amount of manipulation was required to finish many of the sections This viewpoint appeared to be justified from the fact that no surface scaling occurred on the experimental pavements placed by other States with the same equipment

Conclusions Definite conclusions drawn from the investigation referred almost exclusively to the mixtures and the resulting concrete

It was fairly well established that concrete, though unworkable by ordinary means of placing, follows the same laws as concrete of ordinary workability when made plastic by vibration during the process of placing In other words, vibration imparts essentially no new property to the concrete and the water-cement and voids-cement ratio laws still hold good

The advantages of vibration from economic considerations were established It was found entirely feasible to produce concrete of higher strength than standard at about the same cost as standard by reduction of the amount of mixing water and adjusting the proportion of fine to coarse aggregate, also to produce concrete of standard strength at lower cost than standard by maintaining the same water-cement ratio and increasing the amounts of the aggregates in proper proportions

With reference to the first mentioned advantage, it was not determined definitely from the investigation how much increase in strength above that of the standard mixtures may be obtained by reducing the water-cement ratio With reference to the second mentioned advantage, however, a reduction of the amount of cement from that used in standard mixtures of about 15 per cent was obtained without sacrifice of strength Obviously it would also be possible to obtain some increase in strength and save cement at the same time.

It was realized at the time the investigation was under way that it had certain shortcomings and that the manner of conducting it precluded proper study of the practicability of the machine This was indicated by the statement in the conclusions "that the vibratory finishing machine should be further employed with mixtures suitable for it" The opportunity to do so, however, did not occur until September, 1936

INVESTIGATION OF 1936

The results of the investigation of 1932 were not in themselves considered sufficient justification for specifying this method for finishing concrete pavement

There was the question of surface scaling, and while it was never seriously believed that this was an inherent result of the vibratory method, there was still some possibility that it might be

The chief concern, however, was the fact that there was insufficient information as to the practicability of the equipment when operated under job conditions At any rate, it was felt that the equipment should be given a further test on a regular contract job using only mixtures which were strictly suitable for it

An investigation of this nature was undertaken in 1936 The United States Bureau of Public Roads was invited to cooperate and did so in permitting the work to be done on a Federal-aid section, assisting in planning the investigation, and furnishing some of the auxiliary testing equipment

A contract which was awarded during the fall of 1936 specified the use of a vibratory finishing machine and also the use of separated sizes of coarse aggregate The construction work was started so late in the season, however, that it was not only necessary to deal with conditions which always result from late construction, but also to rush construction operations This was at first thought to be unfortunate, but in reality it provided an excellent test of the practicability of the finishing machine In the case of some of the test data, however, the lateness of the season was a handicap, because of the inadequate and variable curing conditions which resulted from low temperatures

The experimental section was designated as Federal-aid Route 142, Federal aid Project 339, Section 34, Sangamon-Menard Counties It is located about 11 miles west of Springfield and extends 5½ miles in a northerly direction from its junction with State Bond Issue Route 125

OBJECT OF INVESTIGATION

In addition to proving or disproving the practicability of the vibratory equipment, another object of this investigation was to determine whether the concrete could be consolidated satisfactorily around all-metal air-chamber expansion joints and the all-metal contraction joints by the vibratory screed without damage to the joints, also to study the use of separated sizes of coarse aggregate, to study thoroughly the concrete obtained, and to make observations from time to time of the finished surface

DESCRIPTION OF FINISHING MACHINE

The finishing machine was a self-propelled unit designated for vibratory finishing It was equipped with reciprocating screeds, the forward one being 18 in wide and the rear screed 12 in wide

Three high frequency electric vibrators were mounted on the front screed, one in the center and one 5 ft from each end The rear screed carried no vibrator units

The bottom plate of the forward screed was curved upward and connected with the vertical plate, the radius of curvature being 2¼ in, thus forming a bull nose which crowded sufficient concrete under the screed to provide for the increased consolidation due to the vibration

A gasoline powered motor-generator unit was mounted on the finishing machine to provide power for the vibrators The frequency of vibration, as determined with a vibrating reed tachometer, ranged from 3,950 to 4,150 impulses per minute, the latter value being that obtained when the screed was resting on the forms and not in contact with the concrete When the screed was pushing a good sized load of concrete, the frequency of vibration ranged from 3,950 to 4,075 impulses per minute The finishing machine weighed about 1,000 lb more than the standard machine, its total weight being 10,700 lb.

Figure 1 is a view of the equipment in operation, showing the front screed and the vibrators.

While there was no essential difference in design of the machines used in the 1932 and 1936 investigations, it should be remembered, in any comparison of data between the two, that the finishing machine used in 1932 carried four vibrator units on the front screed and two on the rear, while that used in 1936 carried three vibrator units on the front screed and none on the rear.

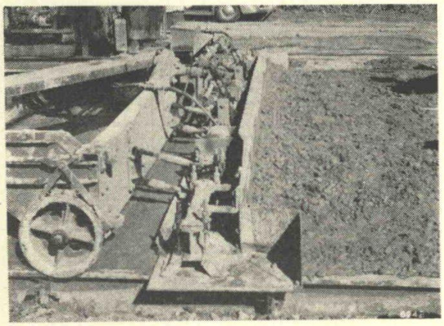


Figure 1. Finishing Machine in Operation. Showing Front Screed with Vibrators

DESCRIPTION OF PAVING PLANT

Outside of the different type finishing machine and an extra crane and bin, the equipment was the same as that regularly used on most paving jobs. A standard 1936 model 27-E paver was used.

MATERIALS

Gravel was used for one-half of the job and crushed stone for the other. The only other deviation from standard practice, was the requirement that the coarse aggregate should be furnished in two sizes, one ranging from 2 in. to 1 in. and the other from 1 in. to ¼ in. These sizes were recombined during the proportioning operations.

The physical properties of the aggregates are shown in Table 1. The cement used was a standard portland cement.

CONCRETE MIXTURES

Standard Mixtures: Since the sources of materials had been used in pavement construction for a number of years, the proportions of the standard mixtures were well known, except as to the effect of two sizes of coarse aggregate. For this reason and also to obtain certain test data for comparison, short stretches of pavement were constructed with standard mixtures without the use of the vibrators, the finishing machine when used in this manner being considered equivalent to the standard reciprocating machine. A total of 1,196 ft. of this mixture was placed using gravel coarse aggregate and 1,709 feet using crushed stone coarse aggregate.

Mixtures of Yield Equal to Standard: In order to determine to what degree increased strength could be obtained with-

TABLE 1

COARSE AGGREGATE

Size	Passing sieve—per cent						Voids per cent	Sp. gr.
	2″	1½″	1″	¾″	½″	No. 4		
Crushed Stone								
2″ to 1″	100	61	10	2				
1″ to ¼″			100	87	27	1		
Combined	100	86	69	57	18	1	40.0	2.73
Gravel								
2″ to 1″	100	68	3	1				
1″ to ¼″		100	98	67	30	1		
Combined	100	87	60	41	18	1	37.5	2.70

FINE AGGREGATE

Passing sieve—per cent					Sp. gr.
No. 4	No. 8	No. 16	No. 50	No. 100	
98	77	62	8		2.65

out greater expenditure for materials, one mixture was designed to produce exactly the same yield as the standard mixture, the proportion of sand to coarse aggregate and the amount of mixing water, however, were reduced to obtain the mixture most suitable for the vibratory equipment A total of 6,264 ft of this mixture was placed using gravel coarse aggregate and 5,288 ft using crushed stone coarse aggregate

Mixtures of Strength Equal to Standard To determine the amount to which the yield could be increased, and thereby save cement, without reduction in strength, one mixture was designed to produce the same strength as the standard mixture, the amounts of the aggregates, however, were increased and the proportion of sand to coarse aggregate was adjusted to obtain the mixture most suitable for the equipment The water-cement ratio was kept essentially the same as in the standard mixture A total of 5,560 ft of this mixture was placed using gravel coarse aggregate and 6,964 ft using crushed stone coarse aggregate

Proportions In order to start the work with a minimum amount of experimentation the proportions believed to be about correct were estimated for each of the mixtures These were then adjusted until the most suitable proportions were obtained

Approximately correct proportions for the standard mixtures were known from previous experience In estimating the proportions for the mixtures to be placed by vibration, the data from the investigation of 1932 were used as a guide, taking into account the difference in the number of vibrator units used in the two investigations and assuming that somewhat greater plasticity would be necessary because of the metal joints around which the concrete had to be consolidated Table 2 shows the proportions originally estimated and those which were determined through experimentation to be best suited, these proportions were used for the greater part of the job

The proportions used were nearly the same as the estimated proportions, except in the case of the vibrated mixtures designed for standard strength, in which somewhat smaller amounts of coarse aggregate produced considerably more satisfactory mixtures Comparing the cement factors of the vibrated mixtures designed for standard strength with those of the standard mixtures, it is seen that the cement was reduced about 11 per cent for the gravel mixture and nearly 9 per cent for the crushed stone mixture, whereas a reduction of 12 per cent was assumed in the estimated proportions

The degree to which the use of separated sizes of coarse aggregate may have affected the yields obtained could not be studied effectively in this investigation Data relating to this are given in the Illinois Division of Highways', Bureau of Materials, report entitled, "A Study of the Segregation of Coarse Aggregate and the Use of Divided Coarse Aggregate for Its Prevention."

In general, 40 per cent of the larger size and 60 per cent of the smaller size, or the 40-60 combination, produced the best results for the gravel mixtures, while the 35-65 combination produced the best results for the crushed stone mixtures

That the use of separated sizes of coarse aggregate provides a desirable latitude in proportioning was clearly shown on one occasion when a 40-60 combination gravel mixture became unsatisfactory and a change was immediately made to a 30-70 combination Investigation showed that the difficulty was caused by segregated material from one of the stockpiles When the next shipment of aggregate arrived, the 30-70 combination became unsatisfactory and it was necessary to resume the 40-60 combination Had it not been for the separated sizes of coarse aggregate, the contractor would either have had to reduce the amount of coarse aggregate or place an unsuitable mixture for the period involved, possibly with an increase in the amount of mixing water

CONSTRUCTION PROCEDURE

An engineer was stationed at the plant to control the proportioning of the mixtures It was his duty to see that the correct amount of each material was measured for each batch and to determine the amount of water to be added at the mixer

Close coordination was necessary between the proportioning engineer and the engineer at the mixer and, since it was necessary for the proportioning engineer to be at the plant most of the time, an engineer from the Bureau of Materials was assigned to assist him at the beginning of the work and at such times thereafter as major changes in the mixtures were made, which necessitated some experimentation before the most desirable proportions of the materials were determined

For a description of the tests required and the duties of the proportioning engineer, reference should be made to the "Manual of Instructions for Proportioning Engineers," March, 1935

The materials were transported to the mixer in trucks having a capacity of two batches The cement required for each batch was placed on top of the aggregates and the bags were not dumped until just before the materials were emptied into the skip of the mixer Each batch contained the exact amount of materials for 29 7 cubic feet of mixed concrete and was mixed for 60 seconds after all of the materials were in the drum

The water measuring device on the mixer was calibrated before the beginning of the work and set to deliver the correct amount of water over a greater range than was to be used

In all respects, care was taken that no error should enter the investigational

TABLE 2

Item	Standard mixtures		Vibrated mixtures			
			Standard Yield		Standard strength	
	Gravel	Stone	Gravel	Stone	Gravel	Stone
Originally Estimated Proportions						
Cement (lb)	94	94	94	94	94	94
Sand (lb)	220	220	205	205	235	235
C A (lb)	369	346	394	371	462	434
Water (gal)	5 40	5 40	4 90	4 90	5 45	5 45
Yield (cu ft)	4 77	4 63	4 77	4 63	5 44	5 27
Cement Factor	1 415	1 458	1 415	1 458	1 241	1 281
Proportions Used						
Cement (lb)	94	94	94	94	94	94
Sand (lb)	220	220	205	205	235	235
C A (lb)	365	354	391	380	448	414
Water (gal)	5 42	5 52	4 92	5 12	5 42	5 55
Yield (cu ft)	4 75	4 66	4 73	4 67	5 34	5 11
Cement factor	1 421	1 448	1 427	1 445	1 264	1 321
Slump (in)	2 4	2 0	1 1	1 1	1 0	1 1

Note The cement factor is barrels of cement per cubic yard of concrete work from inefficiency in control of the proportioning

Placing of Concrete The placing of concrete began September 24 and was finished November 18

The vibratory mixtures were mixed, deposited, and spread in the same manner as the standard mixtures, except for certain modifications introduced as the work progressed The mixer was operated outside of the forms except where it was impracticable to do so

It was found that the mixer operator, by pushing the bucket toward the end of the boom while the concrete was being discharged, could aid materially in spreading the relatively dry mixtures Also, the concrete could be spread across the centerline without materially affecting the longitudinal center joint and tie bars

This method of spreading was at first thought unsatisfactory because some of the larger particles of aggregate were separated from the batch when it was dumped and these tended to deposit themselves against the side forms and joints, but this was remedied by depositing concrete with shovels along the forms and joints in advance of spreading it with the bucket The best edges were obtained when the concrete was shoveled with a turning motion in such a manner that the concrete in direct contact with the shovel would come next to the side forms No spading was done along the side forms with the mixtures placed by vibration

In placing the standard mixtures, the contractor employed three puddlers to distribute and level the concrete and two side form spaders When the vibratory placing was started, the two form spaders were not needed, but it was soon found necessary to use them as puddlers, as three men could not handle the concrete, which was not only drier than the standard mixtures but also contained a considerably greater amount of coarse aggregate

The standard mixtures were of 2-in slump or slightly more, while the vibrated mixtures were of 1-in slump or slightly more The 1-in slump for the latter seemed to be about the lower limit, not only from consideration of proper placement around the transverse joints, but also because of the fact that concrete of lower slump could not be discharged properly from the mixer bucket

The mixtures designed for the same yield as the standard mixtures were somewhat sticky because of the relatively richer mortar A reduction of 0 5 gallon per bag of cement in mixing water was obtained for the gravel mixtures, and about 0 4 gallon per bag of cement for the crushed stone mixtures, which was expected to increase the strength over the standard mixtures

The mixtures designed for the same strength as standard mixtures were better suited for the equipment than those of standard yield because of less stickiness They were placed with about the same amount of mixing water per bag of cement as the standard mixtures and were expected to result in about the same strength The saving in cement, about 11 per cent for the gravel mixture and about 9 per cent for the crushed stone mixture on the basis of the amounts used in the standard mixtures is an item of considerable importance

The percentage of sand in the total absolute volumes of the aggregates provides an excellent comparison between the standard and vibrated mixtures For gravel and stone coarse aggregates, respectively, these percentages were for the standard mixtures, 38 0 and 39 0, for the vibrated mixtures designed for standard yield, 34 8 and 35 7, and for the vibrated mixtures designed for standard strength, 34 8 and 36 9

What has been said in regard to placing of the concrete applied particularly to the mixtures after they had been adjusted to produce the most satisfactory results, these being in general the gravel mixture of the 40-60 coarse aggregate combination and the crushed stone mixtures of the 35-65 coarse aggregate combination, the first figure referring to the percentage of the larger size and the last to the percentage of the smaller size coarse aggregate Some experimentation was made in the case of all of the mixtures before these combinations were adopted to see if some other combinations would give better results When such experimentation was made, the placing of the concrete did not always proceed as satisfactorily as described, though extreme limits in proportions were not attempted and all of the mixtures may be said to have been satisfactory for the equipment, and good results were in general obtained Such experimentation was confined to a minimum and does not represent much of the mileage constructed

The finishing machine made at least two trips over the freshly deposited concrete, but in some cases three trips were required The vibrators were used only during the first trip, during which the finishing machine was operated in low gear

The operations involved in finishing the surface were essentially the same as under standard practice, including the use of a longitudinal float, longhandled floats, belting and brooming Curing was accomplished by 24-hour wet burlap application and thereafter calcium chloride surface application or, when it was too cold for this method, by dry straw covering

The number of irregularities of the surface exceeding ⅛ in in 10 ft, found on the pavement where the vibratory action was employed, was no greater than found on that placed by standard methods The great majority of these were confined to the pavement placed with crushed stone mixtures, indicating that these were perhaps as a general rule slightly harsher than the gravel mixtures However, the crushed stone concrete happened to be placed at the more difficult places, such as on grades, in cuts, and on superelevated curves

Side Forms In general the subgrade was good, since the location followed an old road Side forms of 6-in base were used, but these apparently constituted a more or less definite limitation on the vibratory method of placing the concrete and might have been entirely unsatisfactory on a poor subgrade

Rocking of the side forms occurred to such a degree that it was considered impracticable to operate the finishing machine in second gear with the vibrators in action This, together with time consumed in stationary vibration at transverse joints, left only sufficient time for a second trip over the concrete with the finishing machine in second gear and the vibrators turned off, thus limiting the amount of vibration when keeping pace with the mixer However, under this procedure, no appreciable settlement of the forms was noted, provided they were correctly set and well tamped Under less favorable conditions, forms with wider base would undoubtedly be necessary, even in the case of minimum vibration

A mechanical form tamper was used on a part of the job A comparison, by means of precise levels taken on the forms, between hand tamping and mechanical tamping showed a great improvement resulting from the latter The settlement of the forms was small and regular where the mechanical tamper was used, while an irregular settlement occurred when the forms were tamped by hand Mechanical tamping also prevented the forms from rocking during operation of the finishing machine

Transverse Joints One feature of the present pavement design, the use of all-metal air-chamber expansion joints and all-metal sealed contraction joints, was from the beginning expected to cause some difficulty. These were spaced at 30-ft. intervals, every third joint being an expansion joint. The load transmission feature of these joints consisted of relatively short dowels through the joint working in sleeves provided with wing anchors set in the concrete. These load transmission units were spaced at 20-in. intervals along the joints.

The method of consolidating the concrete around these joints was developed through trial. Various methods were tried and, as soon as the concrete had set sufficiently, a joint was opened and the results examined. This was continued until a method of procedure was developed which insured thorough compaction of the concrete around the load transmission units and against the sides of the joints without collapse of the joints, infiltration of mortar, or damage to the copper seal. The method as finally developed and found entirely satisfactory was as follows:

Concrete was first forced directly under each load transmission unit by shovels. It was then shoveled against both sides of the joint up to the horizontal flange of the copper seal, taking care that no coarse aggregate particles were left on top of the copper seal flange. The best results were obtained when the concrete was shoveled against the joint with a turning motion of the shovel in such a manner that the concrete next to the blade of the shovel was deposited against the joint. Figure 2 shows the concrete deposited along the sides of an expansion joint.

The front screed was stopped about 3 in. from the joint and permitted to vibrate for about one minute. It was then raised and set directly over the joint and stationary vibration was again applied for about one-half minute before proceeding into the next panel. Figure 3 shows the finishing machine in position for stationary vibration at an expansion joint.

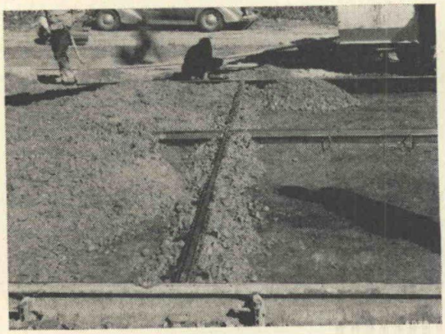


Figure 2. Concrete Deposited Along the Sides of an Expansion Joint

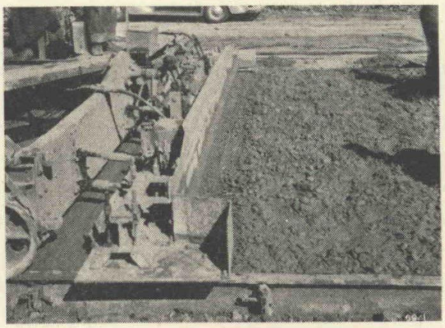


Figure 3. Front Screed of Finishing Machine in Position for Stationary Vibration at an Expansion Joint.

Regardless of the care exercised in keeping coarse aggregate particles off the top of the copper seal, it was impossible to prevent some such particles from being carried forward by the screeds and damaging the seal. In the past, installation bars of various designs had been used and other precautionary methods had been taken to protect the copper seal, but many were impracticable. The vibratory finishing machine still further complicated the problem of protecting the copper seal, because the action of the screed on any particle of coarse aggregate which lodged on top of the seal produced a hammering effect on the particle.

Installation bars for contraction joints were provided by the manufacturer; these gave adequate protection to the seal but were unsatisfactory from a structural standpoint. These bars had wide flanges and they could not be removed after the joint was installed without bending them because of the suction between the bar and the concrete. Furthermore, the space left over the joint was too wide and had to be partially filled.

With cooperation of the contractor, installation bars without flanges were made, which gave a little more clearance for the screed; these proved entirely satisfactory.

The copper seal of expansion joints was protected by premolded bituminous caps. These also suffered damage in the same manner as the copper seal of the contraction joints, though not to so great a degree, and it was decided to try installation bars for their protection.

The manufacturer of the joint submitted installation bars of $\frac{1}{8}$-in. material pressed into the form of a channel conforming exactly to the shape of the bituminous cap. These proved very effective for protection of the cap and for keeping it clean. The only objection was some disturbance of the concrete adjacent to the cap when the installation bar was removed from the expansion joint.

TEST SPECIMENS

The test specimens consisted of 6 by 6 by 30-in. beams and 6 by 12-in. cylinders cast from the concrete at the time of placing, $4\frac{1}{2}$-in. cores drilled from the completed pavement, and special test sections constructed from time to time as an integral part of the pavement and later removed for testing purposes.

Beams and Cylinders: Four beams and eight cylinders were made during each day's construction. When the standard method of placing was employed, the specimens were made by standard methods; and when the vibratory method of placing was employed, the specimens were vibrated for 15 sec. this period having been decided upon from preliminary laboratory tests.

The apparatus used for vibrating the test specimens consisted of a vibrator unit fastened to the bottom of a plank, which was suspended by straps from a trestle. On the top of the plank, provision was made for securely clamping either one beam mold or two cylinder molds. The current for operating the vibrator was obtained from the motor-generator unit on the finishing machine. Figure 4 is a view of the apparatus in operation.

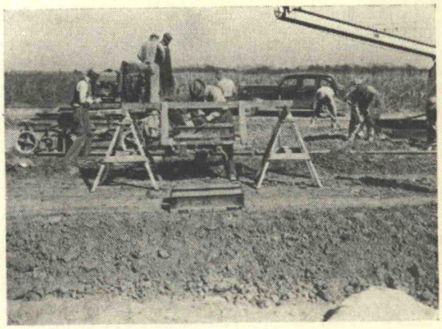


Figure 4. Apparatus for Vibrating Beam and Cylinder Test Specimens

Due to the variation in curing conditions anticipated in late fall construction, the beams and cylinders were transported to the laboratory at Springfield as soon as possible and cured in the moist room. They were from 30 to 36 hours old when this curing was started. They were tested at the ages of 3, 5, 7, 14 and 21 days. An exception to this procedure was made in the cases of the beam specimens from the special test sections which were cured alongside these special test sections and tested with them.

Cores: When the concrete had reached an average age of about 5 months, 144 cores were drilled from the pavement.

These were used for the purpose of studying the crushing strength of the concrete, its absorption, weight per cubic foot, specific gravity, degree of honeycomb, and resistance to alternate freezing and thawing.

Twenty four cores were drilled from each of the gravel and crushed stone mixtures considered the most satisfactory for the vibratory method of placing and also from the standard mixtures. They were taken from well distributed locations and about one-third of them were taken at what was considered the most critical points of the pavement; namely,

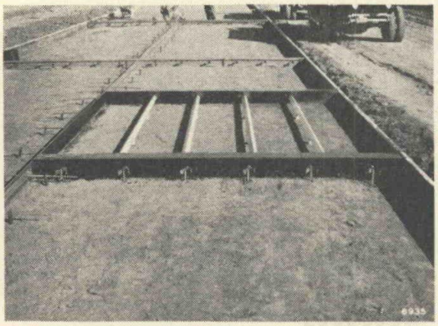


Figure 5. Subgrade Prepared for Construction of a Special Test Section

where the edges of the batches met and near the edges of the pavement, the general opinion being that if a badly honeycombed condition existed at all, it would be found at these points. The cores were tested when the concrete was about 8 months old.

Special Test Sections: In order to examine large samples of the slab and to obtain the flexural strength of the pavement slab itself, special test sections were constructed integrally with the pavement and removed at a later date for examination and tests. These special test sections were constructed and finished in the same manner as the pavement. They extended from the edge to the center of the pavement, were 9 ft. in a longtitudinal direction, and had a uniform thickness of $6\frac{1}{2}$ in. A section was constructed for approximately every 1,500 lineal feet of slab and they were located alternately to the right and left of the centerline. Figure 5 shows the subgrade prepared for the construction of one of the special test sections.

The sections were separated from the rest of the pavement by collapsible wooden headers and four 2 by 2-in. wooden separators were inserted between the headers to crack each section into five slabs, each 24 in. in width. It was thought that the concrete next to the wooden headers would possibly not be representative of the pavement, and provision was made for easy removal of 2 ft. from each end of each slab by making a row of $\frac{3}{4}$-in. holes across each special section 2 ft. from each header before the concrete had set. The ends of the slabs were removed later by means of plugs and feathers. U-bolts were set in each slab to facilitate handling. The arrangement was essentially the same as described in "Public Roads," Vol. 12, No. 6, page 147, August, 1931.

Each test section was placed with two batches from the same truck to eliminate possible variation. No special effort was made in consolidating the concrete beyond that received by the pavement proper, and concrete was not deposited by shovel against the side forms prior to depositing the batches. Three 6 by 6 by 30-in. beams were made from the concrete used in each test section to compare the flexural strength of the usual test specimens with that of the slab. They were kept with the test slabs and cured in the same manner as these until the time of testing.

Upon completion of the paving work, the special sections were removed from the pavement and the place occupied filled with concrete. After removal of the ends, the test slabs were taken to the laboratory, placed outside, and the edges banked with soil. It was thought desirable to let these slabs attain considerable age before testing to minimize the effect of the initial variable curing temperature, and they were not tested until they were about eight months old. Figure 6 shows a special test section ready for removal.

RESULTS OF TESTS

CONSTRUCTION PROGRESS

In connection with the question of the practicability of the vibratory finishing machine, a study of the construction progress on this job is of special interest.

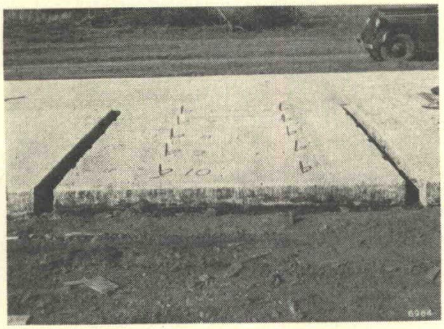


Figure 6. View of Special Test Section Ready for Removal

The average rates shown in Table 3 include all of the pavement laid, except that which could not be classed as representative, such as a wye intersection and one or two short stretches, including most of the pavement placed on the first day before the work was coordinated.

The standard gravel mixture was the first one placed and the 170 linear feet of pavement constructed with it which were considered entirely representative for a study of this nature; on the basis of this distance, the rate of 116.0 linear feet per hour was determined. The remaining rates shown in Table 3, however, were based on more than 90 per cent of the pavement laid with the representative mixes. The lowest rate occurred with the vibrated mixture designed for standard yield. Crushed stone concrete gave slightly lower rates than gravel concrete, although this may be partially accounted for by the fact that the crushed stone mixture, though possibly a little harsher than the gravel mixtures, were placed on the most difficult part of the pavement, containing features such as cuts, fills, and superelevated curves. The vibrated mixtures designed for standard yield were in general too sticky to give the best results. Those designed for standard strength are, from the standpoint of the rates shown, entirely on a par with standard mixtures.

TABLE 3

Mixture	Rate lin. ft. per hr.
Gravel Coarse Aggregate	
Standard	116.0
Vibrated—yield equal to standard	114.7
Vibrated—strength equal to standard	116.8
Crushed Stone Coarse Aggregate	
Standard	110.3
Vibrated—yield equal to standard	107.9
Vibrated—strength equal to standard	114.3

While the vibrated gravel mixture designed for standard strength showed the best average rate, the vibrated gravel mixture designed for standard yield showed the best average rate during a single day. With this mixture, using two $5\frac{1}{2}$-hour shifts, 1,320 linear feet at the average rate of 120 linear feet per hour, or 521 batches of concrete at the average rate of 47.36 batches per hour, were placed in one day. During a 6-hr. period on this same day, the production was as high as 130 linear feet per hour, or about two batches short of a perfect run based on a 62-sec. mixer period.

The average production for the entire job, excepting the wye intersection, was 112 linear feet per hour. The finishing machine was in general capable of handling all the concrete that possibly could be put through the mixer when operated as previously described herein

TESTS OF BEAMS AND CYLINDERS

In spite of the fact that the beam and cylinder specimens were cured in the laboratory moist room, the results obtained from the strength tests did not in heating the mixing water Under these conditions, the erratic results shown in Table 4 might be expected

Rather than to try to make detailed correlation between strengths and initial condition of the mixtures, which at best would be more or less indefinite, it was decided to make strength comparisons only on the basis of the tests of the cores and slabs, which were actually a part of

TABLE 4

TESTS OF BEAMS AND CYLINDERS

Age in days	Gravel mixtures						Crushed stone mixtures					
	Standard		Yield-Std		Str -Std		Standard		Yield-Std		Str -Std	
	No	Str	No	Str	No	Str	No	Str	No	Str	No	Str
Flexural Strength												
3			6	549	4	570	2	546	4	545	6	459
5			6	661	6	651	2	614	8	707	4	660
7			12	729	10	736	4	747	10	757	16	772
14	12	777	14	868	10	786	8	888	12	863	16	895
21			10	913	12	962	2	776	8	916	12	868
Compressive Strength												
3			6	2576	4	2542	2	2162	4	1811	6	1868
5			6	3329	6	3042	2	3393	8	3398	4	3444
7			12	3752	10	3316	4	3963	10	4045	16	3746
14	12	4423	14	4793	10	4103	8	4847	12	4789	16	5025
21			10	5073	6	5084	2	5402	10	4988	12	4843

Number of Tests and Average Strengths in Pounds Per Square Inch

all cases show the anticipated differences probably because of variable factors such as the weather conditions during the 30 to 36 hours that had to elapse before the specimens could be placed in the moist room, the initial temperature of the concrete, and the protection afforded the specimens during their early ages The construction work was started under almost summer conditions, later it became somewhat chilly at night, and after a time it became necessary to heat the mixing water, and finally it was necessary to use a heater on the mixer in addition to the pavement, and which were not tested until the concrete had attained an age at which it would be expected that the effect of the initial variable conditions would be minimized

TESTS OF CORES

About half of the cores were used for strength tests, the other half were used for miscellaneous additional tests Table 5 shows the average specific gravity, percentage absorption, and weight per cubic foot of the concrete produced from the various mixtures These values are in most cases the average of six determinations

It is seen that almost identical results were obtained excepting the standard crushed stone mixture, which appears to be inferior to the rest Either the standard crushed stone mixture was improperly consolidated or the vibrated crushed stone mixtures were benefited sufficiently by vibration to increase the weight per cubic foot more than 2 lb

The strength results, for the sake of correlation with similar data, are shown in Table 6 It will be seen that the standard crushed stone mixture, so far as the tests of the cores are concerned, also showed somewhat inferior strength This will be discussed more fully in connection with the results obtained from tests of the special slabs

Few of the cores drilled showed any unusual degree of honeycomb Only 10 cores showed honeycomb worth mentioning, five each from gravel and crushed stone mixtures Four of these were from standard and six from vibrated mixtures Eight were taken from what was considered the most critical points of the slab, four of these being from standard mixtures and four from vibrated mixtures A few representative cores were sawed longitudinally into halves and the coarse aggregate was in general found to be well distributed in the mortar, leaving no greater degree of voids and air holes than found in the ordinary run of cores In short, the cores were of very excellent appearance, but it may be doubted whether this type specimen is of sufficient size to show clearly anything but severe degrees of honeycomb, especially as far as the bottom of the slab is concerned, as may be seen from the study of the special slabs

Eighteen representative cores were subjected to alternate freezing and thawing tests, consisting of freezing the specimens in the cold room over night under a temperature of 0° F and thawing them during the day in water at 70° F At the present time, after 60 cycles of freezing and thawing, only two specimens show any effects whatever from this test One core of the vibrated gravel mixture designed for standard yield, and one core of the vibrated crushed stone mixture designed for standard strength each show a minute crack, discernible only upon close inspection, which may not be of any particular significance These tests are nevertheless being continued until at least 100 cycles have been obtained

TABLE 5

Mixture	Specific gravity	Absorption, per cent	Weight per cubic ft -lb
Gravel Coarse Aggregate			
Standard	2 37	4 35	147 8
Vibrated—yield equal to standard	2 37	4 35	147 9
Vibrated—strength equal to standard	2 37	4 30	147 6
Crushed Stone Coarse Aggregate			
Standard	2 32	5 02	145 0
Vibrated—yield equal to standard	2 36	4 54	147 3
Vibrated—strength equal to standard	2 37	4 58	147 6

TESTS OF SLABS

The slabs were tested at ages ranging from 231 to 251 days, the average being slightly less than 8 months. They were tested for their flexural strengths and examined to determine the percentage of honeycomb both on the bottom and on the fractured surfaces produced in the strength test. A determination was also made of the percentage of broken coarse aggregate particles in the fracture.

Description of Apparatus and Methods: The apparatus for testing the special slabs was furnished by the United States Bureau of Public Roads. It consisted of a structural steel base frame supporting a transverse rocker bearing and a roller bearing, upon which the slabs were placed for testing. A vertical tension member was attached at the center of each side of this frame by easily removable pins. Fastened at the upper end of these tension members was a transverse steel beam against which the load was applied. The load was applied by means of a ball bearing rachet jack and distributed to the one-third points of the span by means of a structural frame. The load was measured with a micrometer dial by observing the deflection of a calibrated pair of heat treated steel beams inserted between the jack and the beam against which the load was applied. To insure that the roller and rocker bearings would work, metal bars were placed on top of them and molding plaster was used at all points of contact with the specimens to insure even bearings.

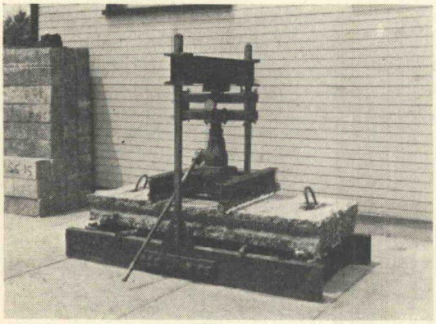


Figure 7. Slab Mounted in Apparatus Ready for Testing

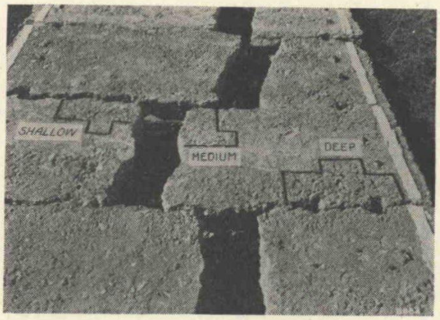


Figure 8. Slab Showing All Three Degrees of Honeycomb

Two weeks or more before testing, the slabs were soaked and thereafter kept wet by ponding water on the surface until the time of testing. They were loaded over the one-third points of a span of 54 in.

Figure 7 shows a specimen mounted in the apparatus ready for testing.

The amount of honeycomb was determined by placing a 4-in. square wire mesh over the surface to be examined and measuring the amount for each square separately. When necessary, the 4-in. squares were subdivided into 2-in. squares. The amount of honeycomb was expressed in terms of the percentage of the total area.

The degree of honeycomb was classified as shallow, medium, and deep. While there was no distinct line of demarcation between these classes, the general rule was to classify any condition where the fine aggregate was not completely covered with cement paste as shallow honeycomb. If the fine aggregate particles and some of the coarse aggregate particles were not covered, the condition was classified as medium honeycomb. If the coarse aggregate particles were definitely not covered by the mortar, the condition was classified as deep honeycomb. Figure 8 shows a slab exhibiting all three degrees of honeycomb The percentage of broken aggregate in the fracture was estimated as closely as possible

Results of the Strength Tests Two special tests sections were constructed for each of the standard mixtures and four for each of the vibrated mixtures and since each special section was divided into five slabs, there were 10 individual tests performed on each standard mixture and 20 on each vibrated mixture Except in case of the first special section constructed, for which no beams were made, there were three 6 by 6 by 30-in beams made with each special section, making a total of three beams for the standard gravel mixture, six for the standard crushed stone mixture and twelve for each of the vibrated mixtures, but three beams of the crushed stone mixture designed for standard strength were lost before testing The average results of the tests of these specimens are shown in Table 6 together with the average compressive strengths of the cores, the latter being the average of twelve individual tests performed at the same age

There is fair agreement, in case of the gravel mixtures, between the strength values obtained for the standard mixture and those obtained for the vibrated mixture designed for standard strength Also, the vibrated mixture designed for standard yield, in which the amount of mixing water had been reduced by one-half gallon per bag of cement, shows on the average about 7 per cent higher strength than the standard strength mixtures, as far as the cores and slabs are concerned The companion beams, however, show discordant results in that the standard strength mixtures show slightly higher strengths

TABLE 6

TESTS OF SLABS, CORES AND BEAMS

Mixture	Strength in P S I			
	Comp	Flexural		
	Cores	Slabs	Beams	Per cent diff
Gravel Coarse Aggregate				
Standard	5381	905	1157	27 9
Vibrated—yield equal to standard	5797	953	1066	11 9
Vibrated—strength equal to standard	5386	877	1133	29 2
Crushed Stone Coarse Aggregate				
Standard	4408	825	1016	23 1
Vibrated—yield equal to standard	6349	944	1075	13 9
Vibrated—strength equal to standard	5277	788	1018	29 1

In case of the crushed stone mixtures, it appears that the standard mixture gave entirely too low compressive strength, while its flexural strength fell fairly well in line with that of the vibrated mixture designed for standard strength The vibrated mixture designed for standard yield, in which the amount of mixing water was reduced by 0 4 gallon per bag of cement, shows an undue gain in strength over the standard strength mixtures Even when the low core strength of the standard mixture is eliminated from the comparison, the gain is about 18 per cent on the basis of core and slab strength The companion beams again show discordant results in that only about 6 per cent gain in strength over those of the standard strength mixtures was obtained

The reason for the low strength of the cores of the standard crushed stone mixture is difficult to explain, especially since the slab strengths did not show a similar tendency. The core strengths would have to average nearly 1,000 pounds higher to be on par with the other standard strength mixtures It will be recalled that the cores from this mixture employed in miscellaneous other tests showed lower specific gravity, higher absorption, and lower weight per cubic foot than the cores of any other mixture, which tends to be in agreement with the low core strengths, but which would scarcely be sufficient to cause the entire deficiency in strength

The beams made in connection with the slabs showed considerably higher strength than the slabs The reason for this lies primarily in the method of test The beams were tested by the so-called cantilever method which is in reality a simple beam test over a 16-in span with the load at the midpoint, restraint being reduced by means of a roller under each end During comparative tests (Report No 31-4, File No 354 0, issued March, 1929), the Illinois Division of Highways found that the one-third point loading gave about 82 per cent of the strength obtained by the midpoint loading using a 24-in span, or, in other words, the midpoint loading gave about 22 per cent higher strength than the one-third point loading In this case, because of the difference in span length between the slab and beam tests, the difference between the two methods would probably not be exactly the same Some of the difference, also, is no doubt due to the difference in thickness of the specimens, the slabs being 6½ in thick and the beams 6 in.

From Table 6, it is seen that the beam strengths for the standard strength mixtures are on the average 28 or 29 per cent higher than the slab strength, excepting the beams of the standard crushed stone mixture which showed a corresponding value of about 23 per cent The vibrated mixtures designed for standard yield, that is, the mixtures in which the water-cement ratio was reduced, showed much lower corresponding percentages, the average being about 13 per cent It would appear then that there is reasonably close agreement between the mixtures within each strength class, and that the beam test within comparable mixtures is a fairly uniform measure of the slab strength However, there appears to be an unreasonable difference in the percentages shown for the two classes of mixtures, and this statement is by no means conclusive

Results of Examination for Honeycomb The vibrated mixtures on the average showed much less honeycomb on the bottom of the slabs than the standard mixtures Honeycomb of any great consequence was not found in most of the slabs when the fracture was examined after testing There was in all instances a large percentage of broken aggregate in the fracture The average percentages for the mixtures are shown in Table 7

The highest percentage of honeycomb was found in the standard mixtures In the gravel mixture, one special section was very badly honeycombed, while the other was in fairly good condition with no deep honeycomb In the crushed stone mixture, both special sections were badly honeycombed

In the vibrated mixtures, conditions of honeycomb, when of any consequence, were generally found in the slabs cast next to the side forms This may be directly charged to the fact that the concrete was not shoveled against the side forms by hand as it was in the pavement proper. Some of the worst conditions found in the slabs, therefore, would in general not be found in the pavement and from this viewpoint the special sections cannot be said to be fully representative of the pavement

Without question, the special sections, however, show a true comparison between the results obtained with the various mixtures Undoubtedly a 2-in slump concrete is not nearly so satisfactory for the standard method of placing as is a 1-in slump concrete for the vibratory method. On the average, the percentage of honeycomb in the standard mixtures was from three to four times as great as that found in the vibrated mixtures

While by comparison the extent of honeycomb found in the vibrated mixtures was much less than that found in the standard mixtures, the appearance of the bottom of the vibrated slabs indicates that a single trip of the finishing machine with the vibrators operating is insufficient for the best results with 1-in slump concrete

It is firmly believed that the amount of honeycomb would have been negligible if there had been sufficient time available to make two trips with the finishing machine over the concrete with the vibrators in action An increase in slump of the concrete, however, would probably not have improved the consolidation since, while only a small range in consistency was employed in the investigation, it was shown that the vibration was not particularly effective on mixtures of 1½-in slump or greater

TABLE 7

Mixture	Broken aggregate in fracture, percent	Honeycomb—percentage of total area				
		In fracture	In bottom of slab			
			Shallow	Medium	Deep	Total
Gravel Coarse Aggregate						
Standard	90 3	0 4	10 9	12 9	3 4	27 1
Vibrated—yield equal to standard	84 1	0 2	4 8	2 7	2 2	9.7
Vibrated—strength equal to standard	83 0	2 9	1 8	3 7	4 3	9 8
Crushed Stone Coarse Aggregate						
Standard	82 5	2 2	6 3	22 7	5 4	34 4
Vibrated—yield equal to standard	86 5	1 5	1 2	1 4	2 7	5 3
Vibrated—strength equal to standard	81 8	2 7	2 0	6 9	2 5	11 4

SURFACE CONDITION AFTER ONE YEAR

A detailed examination of the pavement surface after nearly one year of service revealed few surface defects and no condition which could be classed as surface scaling There were two stretches where rain had fallen on the surface during construction, giving it somewhat different appearance from the rest, and a few locations where some of the surface may have dusted off, but even if this should be classed as scaling, there would perhaps not be ten or fifteen square feet of such in the entire 5½ miles constructed.

CONCLUSIONS

From the data and discussion presented herein, the following conclusions may be drawn

1 The investigation proved that the vibratory method of placing concrete pavement is practical as far as equip-

ment is concerned No mechanical difficulty of any consequence was encountered with the finishing machine, no enlargement of the construction force was necessary, and no additionad equipment was required, with exception of the extra crane and bin needed to handle the separated sizes of coarse aggregate

2 The investigation proved that the vibratory method of placing pavement is practical from the standpoint of progress of the work, an average rate of production of 112 linear feet per hour being maintained for the entire job under late season construction conditions During the most favorable conditions, the rate was higher and often approached closely all that the mixer could handle and still maintain the required mixing period

3 The concrete mixtures used with the vibratory finishing machine were as easily placed and finished as the standard mixtures were with the conventional equipment

4 It was clearly demonstrated that the use of separated sizes of coarse aggregate furnishes a desirable latitude in control of the mixtures

5 It was found that the amount of mixing water used in standard mixtures may be reduced by about one-half gallon per bag of cement, when the mixture is redesigned for use with vibratory equipment by taking out some of the fine aggregate and adding a sufficient amount of coarse aggregate to produce the same yield The increase in strength resulting from this varied considerably between the gravel and crushed stone mixtures and between the various types of test specimens

6 It was found that when the amount of water per bag of cement used in standard mixtures was maintained and the mixture redesigned for use with the vibratory equipment by adding fine and coarse aggregate in proper amounts, a reduction of around 10 per cent in cement content may be obtained without reduction in the strength of the concrete

7 The mixtures were best suited for the vibratory equipment when the amount of sand in the gravel mixtures was about 35 per cent of the total amount of aggregates by absolute volumes and 36 or 37 per cent in the crushed stone mixtures

8 The lower limit of the slump of the mixtures suitable for the vibratory equipment was about 1 in, this limitation, however, being in a large measure due to the transverse joints and the design of the mixer bucket

9 Concrete of 1-in slump may be consolidated very satisfactorily around transverse metal joints by the vibratory finishing machine Installation bars to protect the copper seal or bituminous cap of the joints are necessary

10 The 1-in slump concrete was about as dry as could be satisfactorily discharged and spread by the mixer bucket Possibly some change in design of the mixer bucket would be advantageous where the vibratory type of finishing machine is used

11 Satisfactory edges were obtained without spading when concrete was deposited by shovels along the forms prior to the depositing and spreading of the batches

12 Heavier forms than used on this job would be necessary

13 The amount of honeycomb present in the concrete placed by standard and vibratory methods showed that the vibratory method is far superior to the standard method in consolidating the concrete A 2-in slump concrete placed by the standard method does not give nearly so satisfactory results as a 1-in slump concrete placed by the vibratory method However, a single trip of the finishing machine with the vibrators in action is not believed to be sufficient for best results

14 It is indicated that the standard flexural test of beam specimens used by the Illinois Division of Highways as a

field test gives results which are a fairly uniform measure of the actual slab strength within any certain strength class of the concrete

15 An examination of the pavement surface nearly one year after the construction revealed few surface defects of any kind and no surface scaling

16 Any variation in results between the investigations of 1932 and 1936 is largely due to the difference in the number of vibrator units employed with the finishing machine, and in the limitation as to slump of the concrete imposed by the presence of transverse joints in the 1936 investigation If special test sections had been constructed in connection with the investigation of 1932, thus permitting an examination of the bottoms of the slabs, some of the conclusions drawn from this investigation might have been modified

DISCUSSION ON VIBRATION OF PAVEMENT CONCRETE

Mr Bert Myers, *Iowa State Highway Commission* The work covered by this study is a part of a paving project 7 646 miles long built August 26 to October 22, 1937 The test results reported cover approximately one mile of pavement placed by the Jackson Tube in single day's runs alternated with runs of similar length placed in accordance with the 1937 Standard Specifications of the Iowa State Highway Commission For the sake of brevity the two will be referred to as "Vibrated Concrete" and "Standard Concrete"

The report of the Wisconsin Highway Commission describes the Jackson Vibratory Paving Tube The Iowa Standard Specifications require that all concrete within 18 in of all joints, including the center joint shall be consolidated by means of an internal vibrator The "internal vibrator" used on the standard concrete was a Mall vibrator This tool was not used in the concrete placed with the Jackson Tube

The special provisions requiring the use of the Jackson Tube provided that in the sections to be consolidated by means of the vibratory tube the proportions should be those which would produce concrete with a satisfactory degree of workability for this method of placement, with the provision that the water-cement ratio should not be greater than the average used in concrete placed in the standard manner and that the quantity of cement per cubic yard of concrete should not be less than 1 6 bbl

The dry weight proportions used in the two kinds of concrete were as given in Table 1

The composition of a unit volume of fresh concrete was determined on one 300 ft section of each kind of concrete by observing carefully the quantities of all the materials used and measuring the space filled by taking cross sections of the pavement slab Cross sections were taken at intervals of 10 ft with measurements at one foot intervals across the pavement

The compositions of unit volumes of concrete as thus determined are given in Table 2

Fifty cores were drilled from each kind of concrete A summary of the results of density tests made by weighing in water and weighing in air after being thoroughly soaked in water at laboratory temperature is given in Table 3

The mean deviation from the average of these density determinations expressed as percentage of the average is as follows:

	Percent
Standard Concrete	0.55
Vibrated Concrete .	0 45

One possible explanation for the fact that there was greater variation in the density of standard concrete than in vibrated concrete is that perhaps some of the cores from the standard concrete were taken from areas that had been affected by the Mall vibrator, while others were taken from areas not so affected.

Of the 50 cores drilled from each kind of pavement one-half were sawed in half lengthwise to be examined for evidence of "honey comb" or porosity Two of the cores from the standard concrete showed honey comb extending about one inch up from the bottom of the slab for about half their diameter One of the cores from the vibrated concrete showed similar honey comb A visual examination of the sawed surfaces of the half cores does not show any appreciable difference between the two kinds of concrete as to quantity, size or distribution of pore spaces A more thorough study of this point will be made later

Table 4 gives a summary of the results of compressive strength tests on the 25 cores from each kind of concrete at age 60 days

TABLE 1

	Standard concrete	Vibrated concrete
Cement	1 0 lb	1 0 lb
Sand (sp. gr 2 65)	1 946 lb	2 165 lb
Stone (sp gr 2 58)	2 842 lb	3 162 lb
Water (average)	0 436 lb	0 426 lb
Cement per cu yd	1 71 bbl	1 60 bbl

TABLE 2

	Standard concrete	Vibrated concrete
Cement	0 1226	0 1128
Sand	0 2827	0 2893
Stone	0 4242	0 4340
Water	0 1606	0 1537
Air	0 0099	0 0102
Total	1 0000	1 0000
Total voids	0 1705	0 1639
Total solids	0 8295	0 8361
Void-cement ratio	0 667 cu ft per bag	0 691 cu ft per bag
Water-cement ratio	0 629 cu ft per bag	0 643 cu ft per bag
Weight per cu ft	149 08 lb	149 39 lb

The mean deviation from the average is as follows

Standard Concrete 525 lb
= 7.8 percent of average

Vibrated Concrete 374 lb
= 5.5 percent of average

The evidence presented shows little difference in the quality of the concrete as affected by the difference in the method of placing In fact the differences in average density and strength are so slight as to be considered identical. The vibrated concrete is slightly the more uniform in both strength and density.

The vibrated concrete required 0 11 bbl less cement, 0 0252 tons more sand, and 0 0369 tons more coarse aggregate per cubic yard than the standard concrete

It was noted that placing by means of the vibratory tube required one less man on the crew than was required when the concrete was placed in the standard manner Assuming that cement cost $2 00 per bbl., sand $1 00 per ton, coarse aggregate $1 50 per ton, labor $0 50 per hour, the contractor placed 50 cu yd of concrete per hour and disregarding the difference in equipment costs for the two kinds of vibrators required the vibrated concrete cost $0 15 less per cubic yard than the standard concrete

It should be noted that no attempt was made to determine the harshest or driest mixture that could be placed by means of the vibratory tube The slump of the standard concrete was about 1½ in while that of the vibrated concrete was about ¾ in Both mixtures were easily handled by the methods used

TABLE 3

	Standard concrete			Vibrated concrete		
	Max	Min	Ave	Max	Min	Ave
Density (grams per cc)	2 420	2 358	2 390	2 429	2 365	2 402
Weight per cu ft , lb	151 01	147 14	149 11	151 57	147 58	149 88
Percent of average	101 27	98 68		101 12	98 46	

TABLE 4

	Standard concrete			Vibrated concrete		
	Max	Min	Ave	Max	Min	Ave
Strength, lb per sq in	8140	5210	6751	7620	5740	6806
Percent of average	121	77		112	84	

Mr R B Gage, *New Jersey Highway Department* I was certainly very much interested in the papers which were presented regarding vibration, and I can see no reasons to doubt that they correctly present the value of vibrating concrete used in pavements Mr Jackson, however, I believe was conservative in his estimate of about ten percent increase in compressive strength According to our data we have more than that Unfortunately, the conclusions are all based on compressive strength, but there are other factors than compressive strength that should be considered as indicative of the benefits to be thus secured

We might be eaten up by a lion, or killed by a wolf, but the chances of our thus being eaten or killed are so remote that no one thinks of such a death, yet a microscopic bug comes along and the first thing you know he has you, and you are soon down and out The same is also true of concrete We might have plenty of voids in the bottom of a concrete pavement, but usually they are large, and what damage they have done or can do is practically none We have had voids in the bottom of pavements, and made studies of a lot of them, but the voids have never caused a failure.

However, get a little porous concrete in a pavement and the bugs soon get to work and the first thing you know you will need a new pavement Surface waters will be absorbed if the concrete does not have a certain density Where water can get into a concrete pavement, it can easily get out when conditions are reversed, and every time it goes in and out it does a certain amount of damage, and the concrete sooner or later goes to pieces

With concrete constructed from a pure quartz sand and trap rock, there is nothing in it to be attacked by surface waters, except the cement Both the rock and the concrete sand are durable, yet the concrete goes to pieces The cement has been decomposed—why? The factor we are most interested in is that the life of concrete depends upon its density If we get density, the concrete is going to last, but if we do not, it is going to pieces Cement is only a hydrated silicate or silicates, and it should not be expected to be immune from attack If you will look through the list of the hydrated silicates you will soon discover there is not one that is stable These silicates are soon decomposed, and the residues are oxides These apparently are the only compounds that are stable Since cement is composed of hydrated silicates, we should protect them as much as possible, so that they will not be decomposed by water If the concrete does not have the desired density, the cement or silicates will not be protected, and that is the chief cause of decomposition, with ultimate failure of the concrete

Our experience with concrete pavements has been that if we get the desired compressive strength, we are safe in assuming that we have the desired density, and that we are going to get a durable pavement Generally we have, but how high that compressive strength should be to be perfectly safe, nobody knows We have never yet had a concrete pavement too strong, or too good

Since we revised the portland cement specifications we have added 500 to 1000 lb per sq in to the compressive strength of the concrete at 28-days, the average now being about 6500 lb per sq in The workability of the concrete has also increased, the sheen coating of mixing water on the finished pavements has likewise disappeared, and scaling is practically negligible, compared to what it once was

In New Jersey we have a definite record of where certain cements, sands and coarse aggregates were used with which concrete was secured which has not developed defects in from six to eight years, but in other cases, similar pavements have cracked, peeled and scaled, and are ready to be resurfaced or rebuilt Yet everything was identical in each case, with the exception of the construction methods

We have sand pits and trap rock quarries which have been working for 30 years, so that we know that we have the same materials that produced desirable pavements in one case, and bad pavements in another The only thing that has killed the one, has been the use of excess mixing water With the stronger pavements we are now building, we have had very little, if any scaling, but the methods of construction discussed her today have not been used I am thoroughly convinced that the water-cement ratio is not the thing to use in building pavements If strengths are to be used as a guide in constructing a pavement, why not use enough cement to give the compressive strength required? An engineer when preparing a contract, should know how much cement he wants to use, and should see that it goes into the pavement

With the vibrolithic method of constructing concrete pavements, 200 to 300 lb more coarse aggregate can be used than otherwise, and still a nice surface finish can be secured With an increase in the coarse aggregate, the quantity of cement in the pavement is automatically reduced, but this does not affect to any extent, the composition of the mortar, which, according to our data, is the vital factor The more cement the mortar contains within certain limits, the longer the concrete is going to resist decomposition Our records show that such is the case Why should we monkey with another theory, reducing the cement content to a point where we are not sure the durability of the concrete will not be affected

During the past seven years we have not made a change in the composition or method of fabricating concrete pavements We use a definite quantity of coarse aggregate per bag of cement, and if we find the cement is not working in a desirable manner, we go into the field and find out what the trouble is, and correct it Some of these pavements are now seven years old, yet in some cases the broom marks have not worn off In one particular case, there is not a single transverse crack or broken corner, or one square foot of peeling These results indicate that the methods we are using are producing pavements that will certainly give the service desired and anticipated

MR A A LEVISON, *Blaw-Knox Company* It is interesting to note the development of vibratory equipment as used in France, which was so well described by Professor Crandell, compared with the development of similar equipment in this country It is quite evident that the equipment used for vibratory concrete pavements in France, and I believe that this applies to other foreign countries, does a much more violent and powerful job than the types of equipment that have been used in this country.

I might say that those interested in the manufacture and development of equipment for vibrating concrete for paving work are going to be somewhat perplexed, as to whether to develop equipment that will give the utmost or the ultimate results from the standpoint of improving the density and strength of concrete through the vibratory method, or whether to stop short of that goal and "tickle" the concrete partially to obtain partial results from the vibratory method I, for one, would be very happy to receive suggestions from any of the members here, or especially those that have had experience with the vibratory method of building concrete pavements,—such suggestions would help clear the atmosphere as to the amount of vibration and, incidentally, the type of equipment as affected by the amount of vibration that is desired

In connection with equipment for the vibratory method of building concrete slabs, I think I not only speak for the Company I work for in that connection, but also for others who are developing, planning and thinking in terms of equipment to be used for the vibratory method

PROF C H SCHOLER, *Kansas State College* I was very happy to learn that the French have adopted the practice of ramming concrete Most of our ideas and efforts run in cycles When we first started we placed concrete of low mortar content, with but little water, by means of ramming Gradually we added more water, and with the advent of mechanical mixing and transporting, went to the extremes of very wet and fluid concrete We are now slowly returning to the drier types It seems to me that a pavement is an ideal place in which to try out rammed concrete, and I hope that some of our manufacturers develop equipment which may be used in placing dry concrete on our highways

REPORT OF COMMITTEE ON CHARACTERISTICS OF ASPHALTS

By E F Kelley, *Chairman*

Chief, Division of Tests, U S Bureau of Public Roads

SYNOPSIS

A questionnaire relating to the service of asphaltic road materials was addressed to various organizations and individuals having to do with the use of asphalt in highway construction Ninety-one replies were received and these indicated that failures of asphaltic surfaces due to the quality of the asphalt are prevalent and merit serious attention Failures were reported from all sections of the country and were associated with many types and grades of asphalt On the basis of these replies a program of asphalt research is recommended

During 1937, ten laboratories cooperated in a study of film stripping in bituminous mixtures containing various types and grades of asphaltic road materials Widely varying results with different aggregates indicated aggregate character to be of major importance in connection with film stripping As for the bituminous materials, resistance to stripping increased with consistency and cracked materials were more resistant than uncracked The curing of samples was found to have an important effect on test results Modifications of the test procedure are suggested

Summary Report on Questionnaire

During the latter part of 1936 the committee prepared a questionnaire to be submitted to various organizations and individuals, replies to which it was hoped might serve to define the more important problems having to do with characteristics of asphalts as related to unsatisfactory service results

This questionnaire was distributed by the Director of the Highway Research Board to 189 individuals, including 5 consulting engineers, 4 university laboratories, 48 state contact men, 66 county officials and 66 city officials Ninety-one replies were received

From the standpoint of obtaining detailed information which would be of assistance to the committee in developing a program of work, the results of this questionnaire were disappointing However, certain facts were indicated which it is believed are of sufficient interest to summarize as follows

1 The basic question No 1 was "Have you or your organization noted any failures or unsatisfactory results in asphaltic highway construction or surface treatment under your supervision, which you believe to be due to the use of asphaltic material of poor quality? Out of the 91 replies received, 79 gave definite answers to this question Of these, 34 reported that asphalt of inferior quality had been responsible for failures It seems apparent from these reports that *failures or unsatisfactory results in asphaltic highway construction attributable to the use of asphaltic material of poor quality are sufficiently prevalent to merit serious consideration*

2 *The occurrence of failures or unsatisfactory results from this cause is not localized but is found in practically all sections of the country* Thus, of the 26 states east of the Mississippi, six—Massachusetts, North Carolina, Florida, Alabama, Mississippi and Wisconsin—reported unfavorably and of the 22 states west of the Mississippi, eight—California, Idaho, Arizona, New Mexico, Colorado, Kansas, Missouri and Minnesota—reported unfavorably Twenty states just as widely scattered reported favorably

3 *Trouble with the use of cracked asphaltic products appears to be more*

prevalent than with uncracked products Out of the 34 reports of unsatisfactory results, 17 specifically mentioned cracked or probably cracked asphalts and but 9 specifically mentioned uncracked asphaltic products.

4 *Among the asphalt cements the harder grades appear to be the chief source of such unsatisfactory results as reported* Thus out of 26 unfavorable reports, 21 mentioned asphalt cements of 60 penetration or less

5 *Trouble with the liquid asphaltic products is not limited to any particular types or grades but appears to be more prevalent with cracked than with uncracked products* Out of 25 unfavorable reports, 12 covered cut backs (RC and MC) and 13 slow curing (SC) material Sixteen of the replies specifically mentioned cracked or probably cracked products and 7 specifically mentioned uncracked products

6 *Cracking of the asphalt construction or treatment, and drying out with ravelling or dusting seem to be the principal types of failure attributed to poor quality of the asphaltic products* Out of 51 definite statements of character of failure, 23 specifically mentioned "cracked" and 16 "raveled, dried out and dusted"

Cracking appears to be most prevalent in hot mix construction, 18 out of 19 reports of unsatisfactory results with this type specifically mentioning cracking Both coarse and fine aggregate mixtures were included Raveling, drying out and dusting appear to be the most prevalent trouble in those types of construction and treatment employing liquid asphaltic products

7 *Hardening and increased brittleness of the asphalt were the most frequently observed changes in the asphalt producing unfavorable results* Thus among 22 replies on observable changes in the asphalt, 10 mentioned hardening and 10 brittleness

CONCLUSIONS

From this summary it would seem that the most promising lines of research dealing with control of quality of asphaltic products are:

1 Development or standardization of some rapid laboratory method of determining the relative resistance of asphaltic products to hardening and other changes

2 Correlation of such a test with service behavior of asphalts in the various types of asphalt surface courses

3 Selection of limiting values for such a test which could be specified so as to exclude material likely to prove inferior for any given type of construction

4 Producers could then investigate the possibilities of improvement in quality through selection of raw materials and variations in refining methods

RESISTANCE OF BITUMINOUS MIXTURES TO FILM STRIPPING

A DIGEST OF THE RESULTS OBTAINED IN A COOPERATIVE STUDY CONDUCTED IN 1937 BY THE PROJECT COMMITTEE ON THE CHARACTERISTICS OF ASPHALTS OF THE HIGHWAY RESEARCH BOARD

This cooperative investigation was undertaken to study the resistance to the stripping of bituminous films from hydrophilic aggregates in mixtures containing various types and grades of asphaltic road materials, to determine if the different asphaltic products show any marked difference in resistance to stripping A supply of altered rhyolite of No 8—$\frac{3}{8}$-in size and having definite hydrophilic properties was furnished through the courtesy of the Massachusetts Department of Public Works and a quantity of this was furnished to each cooperating laboratory Each laboratory agreed to make stripping tests according to a standard procedure with this aggregate, and any others that it might desire to study, in mixtures containing as great a variety of asphaltic materials as might be readily available

The instructions furnished, regarding the test method to be used, were as follows

TEST PROCEDURE

A weighed sample of the material is mixed with approximately 5 percent by weight of the asphaltic product under investigation If the asphaltic product is a liquid at normal atmospheric temperature, the mixture is then spread out in a thin layer and allowed to air season overnight

A 50-gram sample of the mixture is separated as nearly as possible into individual fragments or small clumps and placed in a 250-cc Erlenmeyer flask with 175 cc of distilled water

The flask and contents is then placed in the frame of the agitating machine (see page 43, January, 1932, Proceedings, the Association of Asphalt Paving Technologists) and rotated at a speed of 45 r p m for a period of 15 minutes at room temperature At the end of this period the sample is examined for evidence of film stripping and the observation recorded

Rotation of the sample for an additional 15 minutes at room temperature is then made and notation made of any film stripping which may have occurred If little or no stripping has been noted at room temperature, the test is repeated for another period of 15 minutes at 100° F, the water bath being maintained at this temperature throughout the period of test

If little or no stripping has been noted at 100° F, the test is then repeated at 120° F

It is suggested that in the cooperative test, irrespective of the extent of stripping which may have occurred at lower temperatures, the entire procedure involving a total agitation of one hour be carried through

The test is not quantitative in nature and observations as to extent of stripping may vary with individual operators It is therefore suggested that the following general classification be adopted for reporting observations at the end of each period of agitation

1 No stripping
2 Slight stripping
3 Bad stripping

Note It is suggested that if apparently 25 percent or more of the surface area of the aggregate particles has been stripped, the results be reported as bad stripping

A DIGEST OF THE REPORTS SUBMITTED BY THE COOPERATING LABORATORIES [1]

Apparatus In general, the apparatus used conformed to that suggested by the committee Several laboratories made slight modifications such as the addition of heating coils, bath insulation and rate of rotation The variation in rate of rotation was between 38 and 45 r p m and one laboratory which used both 38 and 45 r p m reported no difference in results due to this variable There were some expressions of dissatisfaction with the glass Erlenmeyer flask because of breakage and difficulty of handling the material through the narrow mouth Figure 1 gives a general view of the testing equipment used by one laboratory

[1] Reports were submitted by

1 R R Thurston and B Weetman, Technical Division, Texas Company
2 Joseph Zapata, Wisconsin State Highway Commission
3 J E Myers, Division of Engineering, Department of Public Works, Albany, N Y
4 H P Rue, Bureau of Mines, Laramie, Wyo
5 H Allen and W E Gibson, State Highway Commission, Kansas
6 The Asphalt Institute, New York City
7 H W Skidmore, Chicago Testing Laboratory, Chicago, Ill
8 The Department of Highways, Minnesota
9 D H Jenks, Ashland Oil & Refining Co, Ashland, Ky
10 The Division of Tests, Bureau of Public Roads

Preparation of the Test Sample: One investigator made allowances for solvent and water in proportioning mixtures with cutbacks and emulsions to yield mixtures having 5 percent of actual bitumen. The other investigators proportioned the test mixtures without making allowance for loss of solvent.

Several investigators studied the effect of additional curing beyond that suggested by the committee. Variations therefore covered air curing for periods of 1, 2, 4, 7, 12, 20 and 27 days and oven curing at 140° F. for 24 hours.

Conducting the Stripping Test: Minor variations in the method of conducting the test included: observation of stripping at 1, 3, 5 and 10 minutes by one investigator in addition to the suggested observations at 15-minute intervals; quantitative estimation of the degree of stripping by several investigators in terms of percentage of the total surface stripped; and the introduction, by some, of intermediate arbitrary degrees of stripping between the three degrees suggested by the committee.

Aggregates Used in the Tests: While all the cooperating laboratories used the altered rhyolite (felsite), several included other aggregates in their tests. These additional materials included quartzite, traprock, limestone, gravel, granite and sandstone.

In testing these other aggregates, the laboratories, with one exception, used approximately the same size and grading as that of the aggregate that was furnished. It is believed that the ⅜-in. maximum-size aggregate used in these tests is as large as can be satisfactorily tested with the present apparatus. Larger particles tend to break the glass containers. One laboratory found that the elimination of the fines by washing facilitated coating the aggregate when preparing the mix and also facilitated the observation of the test results.

Table 1 gives the approximate size and grading of the rhyolite and of the other types of aggregates used in the study.

TABLE 1

SIZE AND GRADING OF AGGREGATES

	Percent
Retained on ⅜-in. sieve	0.0
Passing ⅜-in. sieve, retained on No. 4 sieve	61.4
Passing No. 4 sieve, retained on No. 8 sieve	35.6
Passing No. 8 sieve, retained on No. 10 sieve	1.6
Passing No. 10 sieve	1.4

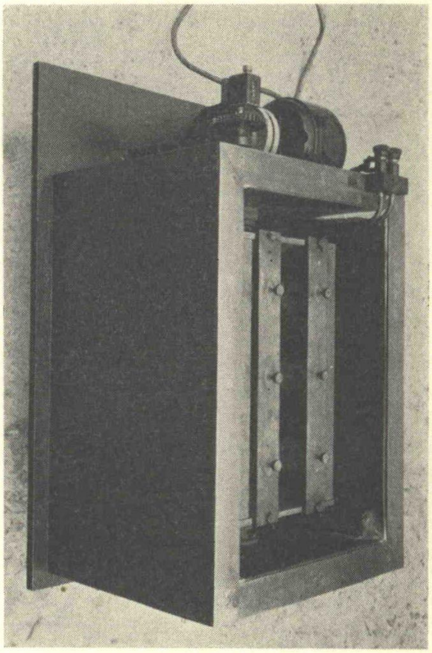


Figure 1. Shaking Machine

Bituminous Materials Used in the Tests: The study covered a wide range in types and grades of asphaltic materials from various sources. These included both straight-run and cracked residuals covering the various grades of SC, MC and RC materials as well as emulsions and the penetration grades of asphalt.

Results of the Tests: The results obtained by the cooperating laboratories were in agreement on the effect of the more important variables These results indicated that the test as outlined was somewhat severe for a number of the liquid asphaltic materials that are now in satisfactory use in dense graded surfacing mixtures since they showed early and in some cases complete stripping One laboratory definitely stated that the test is too severe and another found it necessary to make earlier observations during the first 15 minutes of rotation in order to differentiate between some of the less viscous materials This objection to the test would, it appears, be largely eliminated by providing for considerably more curing of the mixture than that suggested in the original outline of procedure

The wide difference in results obtained with the various aggregates tested by the cooperating laboratories shows the type and character of the aggregate to be of major importance in connection with film stripping Of all the aggregates tested the rhyolite proved the least resistant to stripping In some instances there was also a wide difference in this quality for the same type of aggregate from different sources The quality of the aggregate in its relation to stripping therefore cannot be taken for granted even for any one type of material

For the bituminous materials, particularly those of the liquid type, consistency was shown to have a major effect on the resistance to stripping In general, the resistance to stripping was found to increase with increased consistency. Although there were indications that this also occurred with the penetration grades, the effect of consistency here was of minor importance

The test brought out a substantial difference in the behavior of the cracked and uncracked asphaltic materials The cracked materials possessed much greater resistance to stripping than did the uncracked The difference in this respect was very marked for the slow curing liquid asphaltic materials and to a lesser degree with the cutbacks, emulsions and the penetration asphalts

Of all the bituminous materials tested, the slow curing, residual oils, were the least resistant to stripping while the penetration grades were the most resistant The emulsions and the cutbacks developed resistance to stripping only after they had been allowed to cure thoroughly

The source of the asphaltic materials did not seem to have any appreciable effect on stripping

The overnight air curing suggested in the committee's outline of procedure proved highly insufficient for mixtures containing the liquid asphaltic materials particularly the cutback asphalts and emulsions For the test to be practical for these materials, the test mix must be cured to such a degree that the asphalt approximates the consistency obtained in the mix on the road at the time the pavement is considered finished and ready for traffic As the more viscous, semi-solid materials were not appreciably affected by increased curing they need not be considered in selecting the time or conditions of curing for the test

Both air and oven curing were used in the study Air curing proved ineffective except over long periods of time Oven curing at 140° F for a period of 24 hours or longer appears to be more suitable on account of the shorter curing period required

The results seem to indicate the need for certain modifications in the test procedure In addition to the change suggested for curing of the test specimens, which has already been discussed, removal of the fines from the aggregate by washing seems important The development of a more definite method of determining the amount of stripping and the establishment of limits for differentiating between satisfactory and unsatisfactory resistance to stripping for the different types of surfacing are other important factors which should be studied

The results of this study bear an important relation to the design, construction and field behavior of bituminous surfaces They account for certain types of road failure and they establish certain important principles which should be followed in design and construction

A well-known fact in construction, not always followed, is that in the use of materials which contain a volatile fraction, the conditions and procedure should be such that a major portion of the solvent evaporates before the mix is laid and consolidated Field experience shows numerous failures where cold laid surfaces have been constructed late in the season when conditions were unfavorable to the evaporation of the solvent The almost total lack of resistance to stripping possessed by the materials containing volatile solvents when insufficiently cured strikingly coincides in respect with their field behavior

Type of construction has much to do with film stripping and its effects A dense type of surfacing provides greater mechanical resistance to stripping, and stripping, when it occurs, is not as serious a matter as it is in the more open type of surface which requires a comparatively high degree of bond between the particles to prevent raveling This explains the fairly satisfactory behavior of the western dense road and plant-mix type where slow curing asphaltic materials are used These materials have poor resistance to stripping in the test and cannot be used satisfactorily in the more open type of surfacing The reported satisfactory service of highly hydrophilic aggregates such as the rhyolite used in these tests in conjunction with a dense type of surfacing and a highly viscous asphaltic material, indicates to what degree lack of resistance to stripping may often be overcome by a proper selection of the design and the type and grade of bituminous material

DISCUSSION ON DURABILITY TESTS OF CERTAIN PORTLAND CEMENTS

(Report of Committee on Durability of Concrete as Affected by Cement, Proceedings, Highway Research Board, Vol 16)

MR IRA PAUL, *New York State Department of Public Works* While I agree in principle with the cooperators in this series of laboratory tests on portland cements, that the higher lime cements are more resistant to the action of freezing and thawing, I fail to see just how this test checks up the service records rendered by these cements Some cements similar in composition to those listed in this series that did not show up so well in the laboratory freezing and thawing tests have given us better service in concrete pavements than those that did We have some very outstanding examples of excellent old concrete pavements with cements such as is indicated by sample No 8, and some rather poor ones constructed at a later date as represented by samples No 1, No 10, and No 4

In a paper presented by Professor Lyse, of Lehigh University, before the American Concrete Institute in February, 1935, on, "The Effect of Brand and Type of Cement on Strength and Durability of Concrete," 18 commercial portland cements were considered The composition of the cements used by him were similar in general with such cements as No 1, No 2, No 4, and No 10, as represented in this cooperative series tests, in which the ratio of $\frac{CaO}{SiO_2+Fe_2O_3+Al_2O_3}$ ranges between 2 04 and 2 3 Some of the conclusions drawn by Professor Lyse are rather interesting and read as follows

1 "The durability of concrete containing different cements show considerable variation" (Durability as was measured by the number of cycles of freezing and thawing required to disintegrate the concrete until a loss in weight of 25 percent resulted)

2 "There was no consistent relationship between strength and durability of concrete containing different cements"

3 "No definite relationship could be found between strength and durability of concrete and the chemical compounds for different cements"

4 "The variation in strength and durability of these different cements was probably due more to the method of manufacture of the cement than to any other cause"

To me the most interesting conclusion is the last one quoted.

In all the physical tests I have made on portland cements in mortars or concrete—be it for strength, absorption, permeability, volume changes, or freezing and thawing, I have been unable to tie up any of these tests with the quality of the cement I do not believe we shall get very far in our search of the factors affecting the durability of concrete until we have a better understanding of what determines the quality of the most important constituent in concrete, namely, the cement

The quality of the cement is tied up with its manufacture Our main difficulty has been to evolve a laboratory control method, which will correlate the quality with the manufacture of the cement Several attempts have been made to determine the quality along chemical lines on the basis of oxide analysis, compound composition, and ratios While these attempts have been met with some favor, there have come to our attention several discrepancies, because of erroneous assumptions and other qualifying conditions in manufacture

An investigation is now under way by the American Society for Testing Materials Committee C-1 to check up the value of a "water test" I developed on portland cements, which I believe is related to the process of manufacture

In this connection I might state that cements No 1, No 4 and No 10 of this cooperative series, which had low resistance in the Merriman Sulphate test, showed rather heavy flocs in my "water test" The other samples in this series showed very little or no floc formation in the "water test" Both the Merriman Test and my floc test are chemical tests, and measure not only its sulphate resistant properties but the chemical stability of the cement A cement that is chemically stable is the type which in the long run will give more satisfactory service

MR P H BATES, *National Bureau of Standards* This discussion will cover only that part of the Committee's report which deals with the acknowledged purpose of the tests, namely, the effect of cement compositions on the resistance of the cements to freezing and thawing and "certain other influences" The Committee is to be congratulated on having been able to draw so many conclusions from the data aside from those bearing on the avowed purpose of the research

The data so far as the discussion relating to cement composition is concerned are presented largely in Figures 7, 7A, 9 and 10 These were prepared from the original results as given in certain tables But the graphs just cited and used as a basis for the conclusions are based on certain averages It is the use of these averages that leads one to question the legitimacy and correctness of all of the conclusions made regarding compositional effects If one will consider Figure 6, he can see the "individual" data which were used in some of the averaging He will see that one laboratory has found that freezing and thawing under certain conditions at one age disintegrates the modulus of rupture specimens, while another laboratory carrying out the same (?) tests—but in its own manner—found that the strength ratio was reduced but 0 05, that in another case the ratio was reduced 0 81 according to one laboratory and increased 0 11 by another At three months after 300 cycles, laboratory CU has an average ratio for all cements of 1 04, while laboratory PCA had an average of 0 53 In other words, on the average one laboratory found that the cements were improved by freezing and thawing while another laboratory found the strength was reduced about one-half In spite of such a great spread of results, as one can very readily see in Figure 6, the reader is asked to accept conclusions based upon the question of a procedure of deducing trends from such nonconcordant results

But if the reader does accept these, it is difficult to see how he can accept the statement that Figure 7 indicates a trend showing that an increase of C_3S or the ratio of the percentage of lime to the sum of the percentages of silica, iron oxide, and alumina give better resistance Neither this figure nor any of the figures cited by the Committee as showing trends indicating relations between composition and strength show anything significant other than that there are no trends If in Figures 7, 7A, 9 and 10 one will omit Cements 9 and 10, and in Figure 10 the 14-day results, then it is only possible to deduce that compositional changes have no effect on freezing and thawing results As plotted, for instance, in Figure 9 Cement 8 on the left and Cement 11 on the right (omitting Cements 9 and 10) having the extremes in composition, have so nearly the same strength relations that it would almost take microscopical examination to see wherein there were any differences

Further, the other seven cements lie so nearly on a line joining these two extremes that one is doubly surprised at the Committee's conclusions

Among the "certain other influences" stated as the purpose under "Scope," there was apparently intended some studies of sulphate resistance The Committee with the same broadmindedness that characterized its freezing and thawing procedure, permitted each laboratory to carry out the sulphate tests in whatever manner the laboratory felt inclined to do Hence, four laboratories used three different test procedures Our attention is directed to the fact that "the effect of sulphate solutions was observable on Cements 1 and 10 in which the computed C_3A contents were 14 4 and 12 percent, respectively " Our observations show us that laboratory WU in one set of specimens found Cement 1 was barely adversely affected by the sulphates, while the specimens of Cement 10 were actually improved In another set of specimens the same laboratory found both cements decidely adversely affected But we are curious if the Committee intended us to observe such different deportment in its cryptic statement, "the effect of sulphate solutions was observable " The summary of the SO_3 results would lead one to believe that the Committee connected adverse sulphate resistance with high C_3A content However, laboratory NYSH found Cement 3 with but 1 0 percent C_3A (the least of all the cements) no better than Cement 8 with 7 1 percent and inferior to Cements 5, 6 and 9 with 9 9, 6 6 and 5 1 percent C_3A, respectively. Laboratory WU found Cement 3 (1 percent C_3A) no better than Cement 5 (9 9 percent C_3A) and inferior to Cement 2 (11 4 C_3A) in one set of tests and no better than Cements 5 and 11 (5 2 C_3A) and inferior to Cement 9 (5 1 C_3A) in another It seems that with such contradictory results it is rather difficult to find any relation between the content of assumed C_3A and the sulphate resistance The Committee might have called attention to the fact that Cement 9, found to be the poorest by nearly all tests in its resistance to freezing and thawing, was exceeded by none in its ability to resist sulphate action, while Cement 10, found almost uniformly to be the best in its resistance to freezing and thawing, was about the poorest in its resistance to sulphate action

We hope we will not be considered presumptuous if we suggest that the Committee might have discarded all of its summary and used in its place the following brief statement.

After having carefully studied the data obtained at the several laboratories, your Committee finds itself embarrassed, indeed, it should say more properly, chagrined Having set out "to ascertain the relative resistance to freezing and thawing and to certain other influences of several commercial cements differing in composition," your Committee so much lost sight of its objective that it permitted so many variables to be introduced by allowing each of the cooperating laboratories to follow its own test procedure, that no adequate "findings" can be derived as to the effect of varying compositions on freezing and thawing and "the certain other influences "

PROF C H SCHOLER, *Kansas State College:* Mr Bates' discussion is indeed very interesting I know of no more exhilarating experience than to listen to Mr Bates critically discuss a report The care with which he has studied this report to pick out its weak places and inconsistencies merits the appreciation of the Project Committee Had it not been an important and worthwhile report, I do not believe that Mr Bates would have gone to the trouble to even comment upon it

Mr Bates particularly objects to the conclusions drawn by the Project Committee relating to cement composition, and presented in Figures 7, 7A, 9, and 10 The Project Committee, in presenting the results of this series of tests, as shown in the graphs of Fig 9 and Fig 6, was unquestionably in error in attempting to arrive at the presentation of these results by means of one grand average A reference to Table 10 which shows relative flexural strengths of 1 2 mortars after freezing and thawing at various test periods and varying number of cycles by the various test periods and varying number of cycles by the various cooperating laboratories, will show that without exception, the results from two laboratories are such as to indicate that up to that time the method of test used in that laboratory was not severe enough to be of any significance as indicating the relative durability of the cements If you will observe the relative ratings of these cements at the conclusion of the various test cycles you will note that a variation of 4 or 5 percent between cements is about as high a variation as is shown These variations are no larger than would be expected by the individual specimens of the same cement In dealing with the general problem of durability and resistance to freezing and thawing, small variations in the apparent loss of strength are of no significance whatever So, in general, the results shown by laboratories B P R and C U, and to a considerable extent W U do not show enough discrimination between the various cements to make it worth while to attempt to draw any conclusions The inclusion of the data from these laboratories in the grand average tends to somewhat becloud the issue, but in spite of what Mr Bates says, it has not changed the general line-up as to the relative durability of the cements as shown in the summary of the tables. The variations as shown by these laboratories were so small that any conclusions to be drawn, together with the results from laboratory K S C, would merely be a report of the results secured at the latter institution This is a weakness in the report which the keen eye of Mr Bates did not discover or else was so considerate of the Project Committee's feelings that he did not wish to cause them the embarrassment of realizing that after all, their summary report was largely based on the findings from two laboratories only, P S H and K S. C

If you will refer to that portion of Figure 7 which reports upon the results of the loss from all laboratories, all ages, and all cycles, and refer to Mr Bates' statement that these data indicate that there are no trends, you will note that he has been deceived by a very common ruse in presenting data by graphs By merely enlarging the vertical scale of this figure you are impressed by the strong conclusions that might be drawn from these data

The same thing may be said of Figure 7A in which it will be noted that by changing the vertical scale, a very definite slope will be given to all lines, confirming the conclusions of the Committee

It is the writer's opinion that tests of this type should be interpreted in rather a broad, general way, and for this purpose I prepared a table showing the rating of the different cements at the conclusion of different test periods as determined by laboratory K S C This is then combined into an average rating for the complete period, and this in turn combined with the rating secured by P S. H and P C A These three laboratories being the only ones whose results showed sufficient separation between the various cements and consistent enough results that it might be feasible to try to average them together I have then taken these results and plotted these ratings on the basis of 1 to 10 in descending order against the tri-calcium silicate content of the cement, and against the lime ratio to the other oxides with the results shown on Figure 1 I have then combined the results of the three laboratories and placed them on the same graph with the results from laboratory K S C You will notice the very excellent correlation shown by this graph to the conclusions drawn by the Committee If we were to eliminate one cement from this study, and Mr Bates has suggested that it is perfectly proper to eliminate two in order to conclusively prove your point, the correlation would be almost perfect It is the writer's personal opinion that we are not justified in going to the extreme refinement shown by this graph, that perhaps it would have been perfectly proper to combine the values shown by the high-lime cements, the medium lime cements, and the low-lime cements into three points and again there would be the same unmistakable trend of the data as shown in Figures 7 and 7A

From the inception of the program, Dr Bates was very critical of the fact that the Project Committee had not specified a rigid fixed method for the conduct of the test I am referring to freezing and thawing The writer was equally strong in the contention that probably the most significant facts in the study would be brought out by this variation in the method The conclusions as brought out by the Project Committee amply justify this contention As so frequently happens, the by-products of the investigation are more important than the original information it was sought to attain

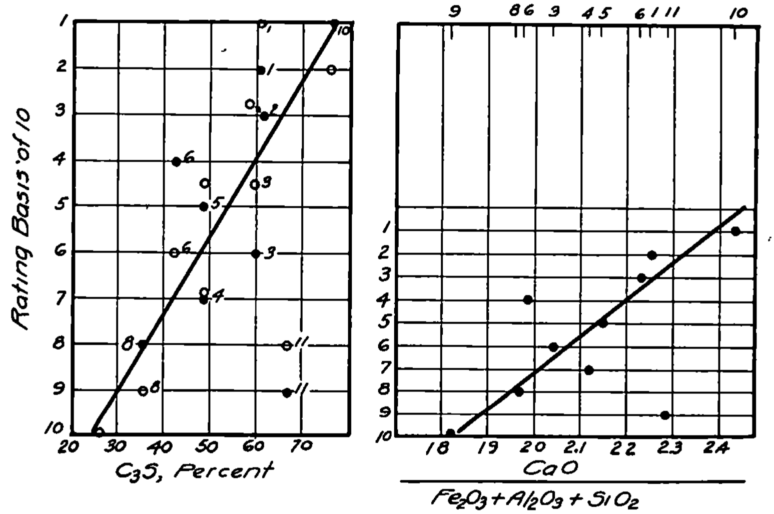


Figure 1

PROF M O WITHEY, *University of Wisconsin* Just before coming to this meeting we measured and inspected the mortar prisms which were made of the various cements used in this program of tests and which have been half immersed, tops up, in pans of water out of doors during the past three winters at Madison, Wisconsin Records from two recording thermometers, one of which recorded air temperatures and the other the temperature within a 6 by 12-in cylinder, indicate that threse prisms have received approximately 80 cycles of freezing and thawing during this exposure The data from these expansion tests have been inserted in a revision of Table 34 of the Appendix to the Committee Report They show that the 1 2 mortar prisms of cements 3, 6, 9 and 11 all of which had low ratios of Al_2O_3/Fe_2O_3 and medium to high contents of Fe_2O_3 and computed C_4AF have expanded much more than the other 1 2 mortars in this program Most of this expansion occurred during the past winter Several of these specimens have expanded over 0 2 percent Certain specimens of cements 9 and 11 expanded so much more on the bottoms than on the tops that the resultant bowing can be easily seen Checking and crazing on the surfaces was also more marked on the 1 2 specimens of these cements than on those of the other cements in the program

The large expansions for mortars of these cements are in agreement with the losses in strengths of similar mortars (Table 9) obtained at the laboratory of the Pennsylvania State Highway Department in the freezing and thawing tests which were conducted on specimens partially immersed during freezing and which we thawed in air This striking agreement in behavior indicates the desirability of partial immersion during freezing followed by thawing in air in making accelerated freezing and thawing tests

The expansion measurements of the 1 4½ mortar prisms of cements 3, 6, 9 or 11 show no excessively high expansions The 1 4½ mortar prisms of cement 4 have expanded materially and disintegrated considerably on the top surfaces and some of the corners of prisms of cements 1, 9 and 10 have crumbled away The maximum individual expansion of the 1 4½ mortar specimens was 0 2 percent, which was less than half of the maximums recorded for specimens of cements 9 and 11 in the 1 2 mortar specimens

Mr R B Gage, *New Jersey Highway Department* What proof do you have that any specimens are made up of the same density Note that density decreased in the properties of the concrete

Professor Withey I have no figures beyond the density figures presented here, but there have been computations made They are shown in the appendix of the report

Mr L C Meder, *California Division of Highways* It has been established through years of work in performing arbitrary tests that only by close standardization of details is it possible to obtain similar results Recently, in correspondence with several laboratories for advice in drawing up purchase specifications for such equipment, this laboratory learned that there were almost as many types of equipment and methods of testing as there were laboratories making such tests We admire the Committee's courage in publishing results that are so wide in range, but wonder if its object was not to emphasize the fact that if any reliance is to be put in the freezing and thawing tests, that test must be standardized

It is apparent that laboratory KSC got greater strength loss after the freezing and thawing cycles than did the other laboratories This laboratory had the most rapid freezing cycle, and the fastest thawing Since the test is designed as an accelerated test, the method giving results in the shortest time is the most satisfactory, if it can be proved that such results correlate with action obtained from natural weathering conditions

We do not feel that one is justified in drawing conclusions when they are drawn from results so inconsistent—to be explicit, laboratory BPR reports a ratio of 1 11 while laboratory PCA reports a ratio of 0 19 on the same cement under the same conditions There are a great number of such inconsistencies throughout the results that tend to cast a reasonable doubt upon the value of the averages

Even if such averages are accepted, the resulting curves do not show a great difference in durability Were Cement 9, which is not a commercially-manufactured cement, eliminated, the resulting curve would be so flat that the influence of C_3S would be almost negligible

The results obtained in the sulphate test show considerably more uniformity between the laboratories involved All agree that Cement 4 with 16 percent of C_3A is the least durable The comparison that laboratory WU draws between 1 2 mortars and 1 4½ mortars bears out further data we have proving that leaner mixes are the less durable

Two of the laboratories used the Merriman Slab Test for evaluating the cements, and obtained somewhat similar results It has been found that the Merriman Test is not always accurate in such determinations It further appears that the total aluminates rather than the C_3A content are more of a factor determining durability when such a test is made in complete immersion in solutions of pure salts

For several years this laboratory has been conducting a comprehensive series of tests to determine the durability of cements exposed to aggressive solutions, and it has been found that certain calculated compounds contribute greatly to the action of a cement under definite conditions of test, but if the conditions are changed, other factors enter giving different results It is our opinion that not only the hypothetical compounds influence the characteristics of a cement, but also certain intangible factors introduced in raw materials and manufacturing processes

We feel that the paper under discussion, instead of closing the tests, should be considered more of a preliminary paper, and open the way for a test with those variables removed which do so much to cast a doubt on the results set forth in the present paper

MR M HIRSCHTHAL, *Delaware, Lackawanna and Western Railway* The writer has found the following points of criticism of the various details of the report

Tests. There are eight types of tests included in the program With more attention to coordination to obtain concordant results and fewer types of tests more valuable results might have been obtained

Procedure · Statement as to age of specimen comparable to parallel ones subjected to freezing and thawing would indicate that the additional time for latter process was not included in age of untreated specimens Such additional time is referred to, in Tables 14 and 15 Was there interpolation of results?

Freezing and Thawing Tests · Variations in equipment and methods do not make for comparable results

Strength Tests Variation in methods here also I call attention to the fact that there is an A S T M standard for compression tests on portions of concrete beams, etc

Data Strength ratios in table do not show strengths of untreated specimens at corresponding age as referred to above

Summary or Results Conclusions unjustified by reason of lack of uniformity of equipment and of methods

(4) Quantitative tests for this purpose are necessary

Relative Resistance of Cements

(2) and (4) are not measures of durability

(5) is inconclusive because of variations

Tables

(3) shows wide variations and inconsistent except Cement No 4

(4) variations in mixing, curing, methods of loading, methods of freezing and various other tests

(8) K S C consistently low throughout

(9) K S C consistently low throughout

(10) K S C consistently low throughout

(11) shows very wide variations for individual cements

(13) no bearing on durability shown

(14) and (15) referred to above

While the conclusions arrived at from the tests seem reasonable, to my mind they are not fully justified because of the fact that they are drawn from results of different laboratories using different methods, different equipment, etc For instance, there is noted a variation in making specimens and obtaining constant consistency in the various laboratories, there is varying equipment for freezing and thawing and variation in times of freezing and thawing, yet the conclusion on this subject is based on tests from laboratories with slow freezing apparatus compared with those from laboratories with fast freezing apparatus It is to be noted also that an examination of the table shows that results from K S C are invariably appreciably lower than those of the other laboratories without any explanation for this condition

Moreover, it is questionable whether freezing and thawing tests are the whole answer to the question of durabilty of concrete or cement

MR H S MATTIMORE, *Committee Closure* In reply to Mr Paul the Committee would say that it is difficult to prove from service records the value of a cement unless *all conditions other than the cement are maintained constant* The Committee agrees with Mr Paul's statement that the strength and durability of concrete are closely related to the method of manufacture of the cement and that we need better control in manufacture If Mr Paul's floc test can be used to determine the durability of concrete made of a given cement, it will indeed be a blessing However, he submitted no substantiating data and the Committee has thus far received none from any source

The Committee well appreciates the variability of the results obtained due to the differences in procedures adopted in the tests made in the different laboratories Indeed, in formulating its program the variations in rates of freezing and in methods of thawing were to a large extent anticipated The Committee fails to share the chagrin so caustically expressed by Mr Bates in the last paragraph of his criticism It is the feeling of the Project Committee that this paragraph is out of place in this discussion From the nature of the program and the data secured it is not logical to omit the effects of the different methods used in conducting these tests The Committee's study of the data indicates that the "slow" rates of freezing accompanied by thawing in water used in three of the cooperating laboratories had small effect on the mortars made of any of the cements Nevertheless, the Committee *presented these data* It did not omit them Furthermore, the Committee asks those interested to examine carefully (1) the trends (in relations of lime ratio and the computed C_3S contents of the cements to the losses in compressive strength shown in Figures 8 and 9, (2) the data on rating of cements according to strengths shown in Tables 24 and 26 (Appendix), and (3) the ratings according to expansion and loss in weight, Tables 29 and 31 (Appendix) before they reject the Committee's conclusions relative to the relation of these quantities to the lime ratio and to computed C_3S

The Committee also wishes to call attention to the similarity in the trend of the expansion data from the tests of the 1 2 mortar specimens exposed to the weather for 3 years at the University of

Wisconsin and the trend exhibited in losses in strengths at the P S H and K S C laboratories during the accelerated freezing and thawing tests of the program

Commenting on two written criticisms presented by Mr Hirschthal, and Mr Meder, their main objection is to some of the findings given in the summary of results in the report, in that they do not feel that in all cases these findings of the Committee are justified from the data in the published report

One statement of some importance is made in the report under Summary of Results—"The data obtained from these tests is too voluminous to report in full in the printed Proceedings" Special reference is made to this as it is quite probable that if all of this data were available to all interested parties, as was the case with the Committee, in interpreting the findings, they would be in a better position to see the viewpoint of the Committee

Variations do exist in results secured from different laboratories This is noted in the published report, and has its effect on the averages, but the Committee thought it advisable to present findings from the averages rather than individual results At the same time the printed report furnishes data enough for those interested so that individual conclusion could be made independent of any findings of the Committee

We feel that the Committee really accomplished something in the results obtained on freezing and thawing as reported by the different laboratories The apparatus used by each laboratory was fully described giving the full information regarding time of freezing and thawing These data compared with the results secured should furnish excellent information for a possible standardization of the freezing and thawing tests Such standardization we all realize is vital at this time, in that, many reports are being submitted regarding the effect of the various cycles of freezing and thawing with little or no data accompanying them regarding the efficiency of the equipment

It is further hoped by the Committee that those interested will make an effort to study the additional tables in the appendix, which can be obtained through the Highway Research Board, and the Committee also solicits further comment so that the full value of the report can be obtained

REPORT OF COMMITTEE ON FILLERS AND CUSHION COURSES FOR BRICK AND BLOCK PAVEMENTS

By J S Crandell, *Chairman*

Professor of Highway Engineering, University of Illinois

SYNOPSIS

The Hocking County, Ohio, brick joint filler test pavement which was built in 1935 was again inspected in September, 1937 by the Project Committee It was found that special pitch No 16 and plastic sulfur "type B No 4" were in questionable condition The two blended asphalts, No 5 and No 6, are apparently in excellent condition

A pavement experiment under way on Route 31, near Logan, Ohio, with brick laid longitudinally, is of interest There was far less breakage during rolling than is customarily experienced, the road is noticeably quieter than is the case with transverse bricks, and since the opening of the pavement fast heavy trucks have not caused breakage A psychological effect noticed is that with the bricks laid longitudinally the pavement seems narrower than a similar width having the bricks in transverse position A report on the longitudinal design will be forthcoming next year

Numerous experiments on brick pavements have been conducted by the Ohio State Highway Department and many of these have been reported to the Highway Research Board The most recent work has been done about 45 miles southeast of Columbus, in Hocking and Fairfield Counties, on the Logan-Lancaster Road Data concerning the fillers used on the project are given in Table 1

The construction of the test road was as follows A 6-in base of 1 $7\frac{1}{2}$ concrete, using natural sand and gravel aggregates, was laid 20 ft wide including two integral 9-in flush curb headers and one 6-in flush center header A brick surface $\frac{3}{4}$-in of 3-in vertical fibre paving brick with projecting side and end lugs laid with a wire cut side up was laid on a $\frac{3}{4}$-in cushion

The base was constructed with impressed transverse contraction joints spaced at 25-ft intervals and a longitudinal key type center joint through the base along one side of the center header There were no joints in the brick surface except in Section No 1 (portland concrete grout filler) where a 3-ply tar paper was used over the contraction and longitudinal joints in the base In addition, this Section had 1-in transverse preformed cork-rubber expansion joints in the brick over some of the contraction joints, spaced at 150-ft intervals

A mastic cushion was used for most of the pavement, however, to enable a further comparison of cushions, two plain sand cushions were also used One with a comparatively fine sand, and the other with a comparatively coarse sand

A calcium chloride solution was used as a filler removal agent on most of the sections, but white-wash was used on part of one section, and a special oil emulsion with plastic-sulfur "type A" White-wash cannot be used as a separating agent with plastic-sulfur fillers

Some difficulties were encountered in the use of some of the fillers

A report made by Mr R R Litehiser after one year's service indicated that fillers No 4, 16, 5, 6 and 3 were apparently satisfactory except that No 3 had an undesirable sulfurous odor Fillers No 18, 12 and 13 had receded into the joints or flowed sufficiently to cause partial failure and filler No 2 had failed badly In September, 1937, two years after its completion, the pavement was again inspected by the Project Committee It was then found that special pitch No 16 and plastic sulfur type "B No 4"

TABLE 1

CONSTRUCTION AND CONDITION SUMMARY

Experimental Brick Filler Project—Logan-Lancaster Road, Hocking-Fairfield Counties, September, 1937

Section No and Length	Cushion	Filler			Principal developments September, 1937
		No	Description	Application	
20[1] 52 ft	Concrete Sand[2]	20	Special Pitch[3], producer designation, F-6	Normal[4], $CaCl_2$ separating agent	Slight exuding in a bubbly formation Shallow checks noted in filler
21[1] 50 ft	Concrete Sand	21	Special Pitch, producer designation, F-7	Normal $CaCl_2$ separating agent	Medium exuding and shallow checks noted in filler
19[1] 59 ft	Concrete Sand	19	Special Pitch, producer designation, F-5	Normal $CaCl_2$ separating agent	Slight exuding and shallow checks noted in filler
16[1] 121 ft	Concrete Sand	16	Special Pitch, producer designation, F-4	Normal $CaCl_2$ separating agent	Practically free from exuding Questionable condition
18[1] 90 ft	First 54 ft Concrete Sand Second 36 ft A C B Mastic	18	Mineral Filled Asphalt 35% to 55% Mineral filler	Normal $CaCl_2$ separating agent	Filler has receded in the joints and has flowed to low side of pavement Holes noted in filler Unsatisfactory
15 95 ft	A C B Mastic	15	Special Pitch, producer designation, Crack and Joint Filler	Normal $CaCl_2$ separating agent	Medium to heavy exuding over entire section
14 94 ft	A C B Mastic	14	Special Pitch, producer designation, Brick Filler	Normal $CaCl_2$ separating agent	Medium to heavy exuding over entire section
12 450 ft	A C B Mastic	12	Natural Lake Asphalt, comparatively low softening point[6] (50°-60°C) and contains 20 to 30% naturally incorporated mineral filler	Slightly difficult to remove $CaCl_2$ separating agent	Filler receded from 1½ to 2 in on high side of pavement and exuded out and over the berm on the low side Unsatisfactory
17 167 ft	A C B Mastic	17	Special Pitch Conforms to the following analysis Softening point, 64°C, bitumen soluble in CS_2, 66%, organic insoluble, 33%, ash, 1%	Difficult to remove Must follow closely after pouring $CaCl_2$ separating agent	Heavy exuding over contraction joint and next to expansion joint in grout section Parts of filler chipped out of joints

TABLE 1—*Continued*

Section No and Length	Cushion	Filler			Principal developments September, 1937
		No	Description	Application	
10 433 ft	First 200 ft A C B Mastic Second 165 ft Concrete Sand Third 68 ft A C B Mastic	10	Asphalt made from a mid-continent crude Higher softening point and lower temperature susceptibility than Std F-1 Softening point, 85° to 96°C Penetration not more than 65	Normal $CaCl_2$ separating agent	Medium to heavy exuding over entire section Amount of exuding not uniform
2 400 ft	A C B Mastic	4	Plastic Sulphur Type "B" premixed combination of approximately 60% asphalt and 40% sulphur	Normal $CaCl_2$ separating agent	Cushion, brick and filler laid in July, 1937, to replace the section shown immediately below
(replaces) 2 400 ft	A C B Mastic	2	Bituminized Grout One part portland cement, two parts special asphalt emulsion and three parts grout sand, by volume	Difficult to get emulsion completely disbursed into mix Excess filler not removed	Large areas where brick are loose and filler is gone Bond loosened between brick and filler and filler crushed Unsatisfactory Replaced in July, 1937, by the above materials
3 450 ft	A C B Mastic	3	Plastic Sulphur Type "A" Two separate parts (sulphur and plasticizing agent, sulphur and powdered silica) were mixed on the job Composition 60% sulphur, 10% plasticizing agent and 30% graded aggregate	Difficult to heat due to non-uniform heat in kettles Temperatures over 320°F appeared harmful Oil emulsion separating agent	No exuding and in good condition with exception of a short section on the east side Filler lifeless and unsatisfactory in this part and an oily exuding has been carried on to the surface Unsatisfactory condition is apparently due to over heating of filler This section has an undesirable odor
9 375 ft	A C B Mastic	9	100% Asphaltic Base Crude Similar to Ohio Highway Asphalt Filler (Std F-1)	Normal $CaCl_2$ separating agent	Medium exuding over entire section Amount of exuding not uniform

TABLE 1—*Continued*

Section No and Length	Cushion	Filler			Principal developments September, 1937
		No	Description	Application	
7 450 ft	A C B Mastic	7	Mastic 60% Ohio Highway Asphalt Filler (Std F-1) and 40% special grout sand (100% pass No 30 sieve and 100% retained on No 100 sieve) by volume Heated separately and mixed on job	Difficult to place due to Segregation Excess filler removed ($CaCl_2$ separating agent) but not reheated	Non-uniform exuding Heavy exuding in joints where sand content is low and no exuding where sand content is high
11 450 ft	A C B Mastic	11	100% Asphaltic Base Crude Lower temperature susceptibility and higher softening point than Std F-1 Softening point, 85° to 96°C Penetration[7], not more than 65	Normal $CaCl_2$ separating agent	Medium to heavy exuding over entire section
13 300 ft	A C B Mastic	13	Pitch (asbestos filled) Conforms to following specification Softening point, 45° to 50°C Bitumen (Sol in CS_2), not less than 70% Organic insoluble, not more than 20% Ash, not less than 7%	Difficult to heat and too viscous to flow into joints Corrected by diluting with 40% hot tar by vol $CaCl_2$ separating agent	Filler has receded into joints and a little has flowed out on curb headers on both sides of pavement Unsatisfactory
1 450 ft	A C B Mastic	16	Special Pitch, producer designation, F-4	Normal $CaCl_2$ separating agent	Cushion, brick and filler laid in July, 1937, to replace the section shown immediately below
(replaces) 1 450 ft	A C B Mastic	1	Portland Cement Grout One part cement and 4 parts grout sand[5] by volume	Squeegeed	Contains areas where brick bond is broken and brick are loose Approximately 10% of brick are loose Replaced in July, 1937, by the above materials
4 450 ft	A C B Mastic	4	Plastic Sulphur Type "B" premixed *combination* of approximately 60% asphalt and 40% sulphur	Normal $CaCl_2$ separating agent	No exuding and no undesirable odor on entire section Questionable condition

TABLE 1—*Continued*

Section No and Length	Cushion	Filler			Principal developments September, 1937
		No	Description	Application	
5 450 ft	A C B Mastic	5	Blended Asphalt Mineral Filled Same as Filler No 6, except that it contains (separately added *in* the process of manufacture) 20 to 30% finely divided mineral matter	Normal $CaCl_2$ separating agent	No exuding on entire section Good condition
6 450 ft	A C B Mastic	6	Blended Asphalt (65% mid-continent base and 35% asphaltic base) low temperature susceptibility type Softening point, 101° to 110°C Penetration not more than 47	Normal $CaCl_2$ separating agent	No exuding on entire section Good condition
8 450 ft	A C B Mastic	8	Asphalt made from a mid-continent crude Meets some specification as Std F-1	Normal $CaCl_2$ separating agent	Heavy exuding prevalent over entire section with increased exuding over contraction joints and low side
A 550 ft	First 150 ft Cushion Sand[8] Second 400 ft Concrete Sand	Std F-1	1934 Ohio Department of Highways Bituminous Filler Softening point, not less than 75°C Penetration, not more than 90	Normal $CaCl_2$ and lime separating agent	Heavy exuding prevalent over entire section

EXPLANATORY NOTES

[1] The first 114 ft (both sides of pavement) and the west side opposite these sections are filled with a mixture of various fillers and are not included in test

[2] Comparatively fine sand, Ohio Department of Highway Specifications, Section M-2 1

[3] Complete specifications on all fillers used are on record at the Ohio State Highway Testing Laboratory

[4] "Normal" indicates that application and removal compared favorably with our standard asphalt

[5] Comparatively fine sand, Ohio Specifications, Section M-2 3

[6] Softening points listed are all for ring and ball method

[7] Penetrations listed are all for 46°C, 50 gr 5 Sec

[8] Comparatively coarse sand, Ohio Specifications, Section M-2 4

were in questionable condition The two blended asphalts, No 5 and No 6, are apparently in excellent condition Fillers 1 and 2 were replaced in July 1937 with Special Pitch No 16 and Plastic Sulphur No 4, because sections 16 and 4 appeared to be in good condition after one year The results of the two-year inspection are given in Table 1

Another experiment is on Route 31 in Ohio, near Logan, where the brick have been laid longitudinally The advantages claimed for this are less broken brick during construction and thereafter, a quieter pavement, no batting, and that the brick will span such transverse cracks as may appear in the base It is too early to make a report on this stretch of pavement which at the present time is highly satisfactory There is however one item to be mentioned which has to do with the psychological effect on the driver rather than with the construction, which is that the pavement seems narrower than a similar one laid with blocks placed transversely The driver, looking down the long rows of bricks to the vanishing point, apparently sees the road narrowing like a wedge It may also be mentioned that at present the pavement is noticeably more quiet than those constructed with bricks laid transversely

When this road was built the inspectors and the roller man reported that there was far less breakage of the bricks during rolling than is customary where they are laid transversely It appears that heavy swift trucks have not been successful in breaking the bricks since the opening of the pavement

There are some new sections of road in Ohio where brick have been laid on powdered asphalt cushions, on Amiesite, and on sand-clay cushions No report is being made on these since they are not yet old enough to merit it The sections are being watched, and as soon as anything definite is learned a report will be forthcoming

Conclusions Two blended asphalts stand out from all the other fillers as being satisfactory after two years' service on a heavy traffic highway Those fillers that caused the sections in which they were placed to fail, have been eliminated from further tests Such sections have been rebuilt, using other fillers

The pavement laid with brick laid longitudinally seems to have some points of superiority over the usual form A report on it will be made next year

SOME FILLER CHARACTERISTICS OBSERVED ON THE HOCKING COUNTY FILLER TEST ROAD

By H Z Schofield, *Director*

Research Bureau, National Paving Brick Association

SYNOPSIS

Added to the results of the planned observations on the Hocking County filler test road is information on the effect of mechanical working on the exuding of asphalt fillers for brick joints

In addition to other benefits a non-exuding filler produces a surface course noticeably more quiet than a surface course with exuded filler

The Hocking County filler test road, both in construction details and in first and second annual inspections of filler conditions, has been reported As might have been expected in an experiment of this scope, the experience has given indications on certain brick joint filler characteristics in addition to the planned filler condition observations

In the preliminary laboratory tests designed to find promising fillers for inclusion in the Hocking County filler test road, the only similated service test for exuding was summer heat application In making inspections of the test road it was found that exuding of certain asphalt fillers was markedly greater in those areas of the brick surface course overlying base contraction joints and other areas in which traffic might be expected to impart a slight movement to the brick units The most pronounced example of this effect was observed on the two sections devoted to Ohio Highway Asphalt Filler Fl, an asphalt of high exuding properties One section in which the brick were laid on plain sand cushion exhibited, after one year's service, a filler exudation of heavy proportions On the other section in which the brick were laid on a mastic cushion (probably more stable than plain sand) the filler exudation was of medium proportions except over each base contraction joint where exuding was heavy In the laboratory, Bencowitz [1] has demonstrated that exuding will take place when thin layers of asphalt are subjected to shearing forces His apparatus consisted of five plates, spaced $\frac{1}{16}$ in apart, and so placed that they moved longitudinally in reference to each other without changing the distance between them Thus the material filling the spaces between the plates was subjected to a kneading action without compression Dr Bencowitz, in discussing these results, indicated that thixotropy or breakdown of structure (with its resultant lowering of the viscosity of the asphalt) may account in part for this effect From these laboratory and service observations it would appear advisable to consider the effect of kneading as well as that of summer temperatures in future laboratory investigations of brick joint fillers

In enumerating the undesirable features of a heavily exuded filler, most prominently mentioned have been the reduction of anti-skid properties and the progressive loss of the filler itself It so happens that, on the filler test road, four non-exuded fillers (two types of plasticized sulfur and two blended asphalts) occur together in a pavement length of about one-third mile Adjoining this is a section of almost one-fifth mile containing exuded Fl asphalt Thus every opportunity has been given for a comparison with the result that to the other desirable features of a non-exuded filler must be added markedly more quiet and smoother riding quality

[1] I Bencowitz, Texas Gulf Sulfur Company

REPORT OF COMMITTEE ON CORRELATION OF RESEARCH IN MINERAL AGGREGATES

R R LITEHISER, *Chairman*

THE RELATION BETWEEN LOS ANGELES ABRASION TEST RESULTS AND THE SERVICE RECORDS OF COARSE AGGREGATES

By D O Woolf

Associate Materials Engineer, U S Bureau of Public Roads

SYNOPSIS

Because all States do not have the same range in quality of coarse aggregates, and one State with an abundance of hard rock will consider certain test values for quality as necessary which could not be used by other States, it must be emphasized that recommendations are subject to change with local conditions In general, however it appears that there is a definite relation between the loss in the Los Angeles test and the service record of materials used in concrete, bituminous construction and surface treatment On the basis of the data available, the following percentages of wear appear to be suitable for use in specifications to control the quality of coarse aggregates Concrete, 50 per cent, Bituminous surfacing, 40 per cent, Surface treatment, 40 per cent

Definite correlations between the loss in the Los Angeles test and the strength of concrete are found, the lower the percentage of wear the higher the concrete strength Definite correlations are also found with the results of a circular track roller test and a test for soft or friable pieces It appears that the Los Angeles test gives accurate indication of the quality of the material under test, and that its use in specifications controlling the acceptance of coarse aggregate is warranted

The proposal to adopt the Los Angeles abrasion test as a substitute for the present standard methods of determining the resistance to abrasion or impact of coarse aggregates has raised the question as to what relations exist between the results of this test and service behavior In 1935 the Committee on Correlation of Research in Mineral Aggregates of the Highway Research Board recommended that a study of this feature be made and a request for information was sent to all highway engineering authorities Considerable information was collected in 1935 and 1936 by the Board, and this was supplemented by additional data secured by the Bureau of Public Roads in 1937

For purposes of discussion the information which has been secured will be classified under two heads, first, that showing a direct comparison between the Los Angeles test result and the service record of the material, and second, that comparing the Los Angeles test result with strength tests of concrete or with wear tests of aggregates which simulate the action of traffic In presenting the data on service behavior effort has been made to present the information from different sources in similar terms in order to permit of ready comparison Also to insure that no error of interpretation has been made, laboratory reference numbers are given, where available, to designate the particular materials under consideration so as to permit ready checking of the service records reported here by the authorities from whom the data were obtained

COMPARISONS WITH SERVICE RECORDS

Data comparing the Los Angeles abrasion test results with service records have been obtained from eight State highway departments and a highway board in Australia. In general, the data as presented in Tables 1 to 9 show the type of material, the Los Angeles abrasion test result, the service record of the material when used in concrete, bituminous pavement, or surface treatment construction, and a laboratory number designating the material. The letter S indicates that the results in service have, in general, been satisfactory, the letter Q that the exact rating is in question and the letter U that generally unsatisfactory results have been obtained. In a number of cases several samples of the same material are shown in the original data from which these tables were compiled. In such cases the value shown in the table is the average of all the values reported. Also the values given as the Los Angeles test result are generally averages of the two gradings permitted in the test method.

Table 1 presents service records compiled by the Florida State Roads Department. With only two exceptions all rock and blast furnace slag with Los Angeles test results of less than 40 percent are classed as being satisfactory for use in concrete, bituminous construction and surface treatment. The two exceptions are a slag with a Los Angeles loss of 28.8 percent which is considered unsuitable for use in surface treatment, and a limestone with a Los Angeles loss of 38.2 and having a rough open texture which is considered questionable or unsuitable for all three types of construction. Gravel from three sources with Los Angeles losses from 41 to 48 percent are classed as satisfactory for use in concrete and bituminous construction, but as questionable for use in surface treatment.

TABLE 1

COMPARISON BETWEEN LOS ANGELES TEST RESULT AND SERVICE RECORD BY FLORIDA STATE ROAD DEPARTMENT

Sample No	Type of material	Description	Los Angeles per cent of wear	Service record		
				Concrete	Bituminous construction	Surface treatment
54673	Rock	Dolomite	14.8	S	S	S
54671	"	Chert	15.2	S	S	S
54577	Slag	80 pound	23.2	S	S	S
54678	"	"	28.0	S	S	S
54676	"	71 pound	28.8	S	S	U
54670	Rock	Limestone	31.0	S		S
54675	"	"	34.8	S		S
54674	"	"	38.2	Q	U	U
54672	"	"	40.9	Q		U
54667	Gravel	Quartz	41.2	S	S	Q
54669	"	"	43.4	S	S	Q
54668	"	"	48.5	S	S	Q

Table 2 presents a comparison between the Los Angeles test result and the service record of materials used in surface treatment work based on data supplied by the Georgia State Highway Board. Both rock and blast furnace slag with Los Angeles losses of 40 percent or less are found to have satisfactory service records.

Data furnished by the Kansas State Highway Commission giving service records for concrete and bituminous construction are shown in Table 3. All materials had satisfactory service records

TABLE 2

COMPARISON BETWEEN LOS ANGELES TEST RESULT AND SERVICE RECORD BY GEORGIA STATE HIGHWAY BOARD

Type of material	Description	Los Angeles percent of wear	Service record in surface treatment
Slag	Copper	14 6	S
Rock	Limestone	26 1	S
Slag	Blast furnace	27 3	S
Rock	Dolomite	38 2	S
"	Granite	40 2	S
Gravel	Quartz	49 1	U
Rock	Granite	61 9	U

The data indicate that satisfactory results were secured with materials having losses as high as 47 4 percent in the case of concrete and 40 8 percent in the case of bituminous construction

Data furnished by the North Carolina State Highway and Public Works Commission regarding the behavior of materials in surface treatment construction are shown in Table 4 All materials found to be satisfactory for this use have percentages of wear in the Los Angeles test of less than 40

TABLE 3

COMPARISON BETWEEN LOS ANGELES TEST RESULT AND SERVICE RECORD BY KANSAS STATE HIGHWAY COMMISSION

Sample No	Type of material	Description	Los Angeles per cent of wear	Service record	
				Concrete	Bituminous construction
27061	Rock	Limestone	30 8	S	S
27039	"	"	31 8	S	S
27069	"	"	32 4	S	S
27070	"	"	40 0	S	S
27071	"	Calcareous sandstone	40 8	S	S
27099	"	Limestone	47 4	S	

TABLE 4

COMPARISON BETWEEN LOS ANGELES TEST RESULT AND SERVICE RECORD BY NORTH CAROLINA STATE HIGHWAY AND PUBLIC WORKS COMMISSION

Sample No	Type of material	Description	Los Angeles per cent of wear	Service record
				Surface treatment
11	Rock	Granite	17 4	S
9	"	Limestone	19 9	S
8	"	"	21 3	S
10	"	Dolomitic limestone	24 2	S
7	"	Granite and diorite	36 4	S
2	"	Granite gneiss	37 9	S
12	Gravel	Quartz—gneiss	41 9	Q
13	"	Quartz	42 6	Q
3	Rock	Granite gneiss	47 1	U
16	Gravel	Quartz	47 6	Q
1	Rock	Dolomitic limestone	47 8	Q
4	"	Pegmatitic granite	50 6	Q
15	Gravel	Quartz	53 0	Q
14	"	"	54 1	Q
6	Rock	Granite	57 0	U
5	"	"	62 4	U

Table 5 presents data furnished by the Ohio Department of Highways showing the service record of materials in concrete and bituminous construction With the exception of gravel from two sources, all materials with Los Angeles losses of less than 35 percent are found suitable for use in the types of construction mentioned No explanation of the behavior of these two questionable materials was furnished

TABLE 5

COMPARISON BETWEEN LOS ANGELES TEST RESULT AND SERVICE RECORD BY OHIO DEPARTMENT OF HIGHWAYS

Sample No	Type of material	Description	Los Angeles per cent of wear	Service record	
				Concrete	Bituminous construction
9	Rock	Limestone	20.5	S	S
10	"	"	23.3	S	S
15	Slag	83.5 lb	27.9	S	S
11	Rock	Limestone	28.5	S	S
5	Gravel	Crushed	28.7	S	S
1	"		28.9	S	S
6	"	Crushed	29.6	S	S
2	"		31.0	Q	Q
16	Slag	72.5 lb	32.0	S	S
13	Rock	Limestone	32.7	S	S
3	Gravel		32.8	Q	Q
12	Rock	Limestone	33.2	S	S
4	Gravel		37.2	Q	Q
8	"	Crushed	38.8	Q	Q
17	Slag	69.5 lb	47.8	Q	Q
14	Rock	Limestone	58.0	Q	Q

TABLE 6

COMPARISON BETWEEN LOS ANGELES TEST RESULT AND SERVICE RECORD BY SOUTH CAROLINA STATE HIGHWAY DEPARTMENT

Sample No	Type of material	Description	Los Angeles per cent of wear	Service record		
				Concrete	Bituminous construction	Surface treatment
A-21778	Rock	Granite	33.4	S	S	S
A-21781	"	"	34.8	S	S	S
A-23260	"	"	41.1	S	S	S
A-22269	Gravel	Siliceous	46.2	S[1]	Q	Q
A-22497	"	"	46.3	S[1]	S	
A-22422	"	"	52.5	S[1]	U	U
A-22421	"	"	53.9	S[1]	U	U
A-21198	Rock	Dolomitic marble	58.2	S		
A-22492	"	Gneiss	67.4	S	U	U
A-22423	Gravel	Siliceous	74.0	U	U	U

[1] For structural service only

Service records for materials used in concrete, bituminous surfacing, and surface treatment are given in Table 6 based on information furnished by the South Carolina State Highway Department These data show materials with Los Angeles losses of 41 percent or less to be satisfactory for use in surface treatment Materials with losses to 46 percent are considered satisfactory for use in bituminous construction and a rock with a loss of 67 percent is considered suitable for use in concrete Four samples of gravel with losses between 46 and 54 percent are considered suitable for structural concrete only

Data furnished by the Texas State Highway Department giving service

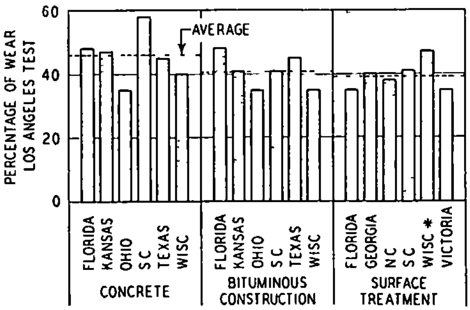


Figure 1 Maximum Percentage of Wear in Los Angeles Test for Material of Satisfactory Service Record

*** Traffic Bound Surfacing**

records for materials used in concrete and bituminous construction are shown in Table 7 With only five exceptions all samples of rock and gravel with Los Angeles test losses of less than 45 percent are found to be satisfactory for use in both types The five exceptions include three limestones with losses of 30 2, 31 6 and 37 5 percent, a siliceous gravel with a loss of 30 6 percent, and a quartzite with a loss of 42 7 percent The limestone with a loss of 30 2 percent is considered satisfactory for use in concrete but questionable for bituminous surfacing The gravel is stated to be laminated, and this may account for its poor service record Two of the three limestones were taken from sources which apparently contained material of variable quality, and it is possible that the low values given represent material of better quality than has been supplied for use All materials with losses above 45 percent are stated to be unsuitable for bituminous construction and those above 51 percent to be unsuitable for use in concrete

Table 8 shows service record data for concrete, bituminous construction, and traffic bound surfacing furnished by the Wisconsin State Highway Department The data are not complete in that service records for all three classes of construction are not shown for a majority of the materials However, all materials with a Los Angeles loss of less than 40 percent are stated to be satisfactory for use in those types of construction where a record is available With the exception of one sample, a dolomite, all materials with losses of 47 percent or less were reported as satisfactory for use in traffic bound surfacing This surfacing was placed largely on town or county roads and presumably carried only light traffic

In Table 9, the service record of materials used mainly in surface treatment construction is given from records furnished by the Country Roads Board of the State of Victoria, Australia[1] All rock and gravel with a Los Angeles loss of 35 per cent or less are stated to have excellent to fair service records, but the Board concludes that a maximum loss of 20 percent should be used for the best surfacing materials

A summation of the foregoing information is presented in Figure 1, with the findings of the different highway authorities grouped according to type of construction Some liberties have been taken in presenting these data in that the values given may include a few materials without satisfactory service rec-

[1] The Los Angeles Abrasion Test, by A H Gawith, *Main Roads*, Vol 8, No 2, Feb 1937, p 54

TABLE 7
COMPARISON BETWEEN LOS ANGELES TEST RESULT AND SERVICE RECORD BY TEXAS STATE HIGHWAY DEPARTMENT

Sample No	Type of material	Description	Los Angeles per cent of wear	Service record	
				Concrete	Bituminous construction
2979	Rock	Rhyolite	16 1	S	S
2992	"	Dolomitic	19 0	S	S
1934	Gravel	Siliceous	20 6	S	S
2951	"	"	22 2	S	S
3699	Rock	Limestone	22 2	S	S
2838	Gravel	Siliceous	24 5	S	S
2815	"	"	25 4	S	S
3557	"	"	25 6	S	S
3754	"	"	25 7	S	S
3723	Rock	Limestone	26 3	S	S
3672	"	"	26 9	S	S
3000	Gravel	Limestone and siliceous	27 0	S	S
2873	Rock	Limestone	27 2	S	S
3698	"	"	27 5	S	S
2243	Gravel	"	29 0	S	S
3092	Rock	"	29 1	S	S
2898	Gravel	Siliceous	30 0	S	S
2767	"	Limestone	30 1	S	S
3670	Rock	"	30 2	S	Q
3538	Gravel	Siliceous	30 6	Q	U
2975	Rock	Calcareous shale	30 8	S	S
2667	Gravel	Limestone	31 3	S	S
3766	Rock	"	31 6	U	U
3013	"	"	32 2	S	S
3823	"	"	33 2	S	S
3668	"	"	34 5	S	S
3783	"	"	35 1	S	S
3724	"	"	36 3	S	S
3803	"	Dolomite	36 3	S	S
3014	"	Limestone	37 5	Q	U
2367	Gravel	Siliceous	37 9	S	S
2824	Rock	Limestone	38 2	S	S
3228	"	"	39 3	S	S
3111	"	Quartzite	42 7	Q	Q
3824	"	Limestone	42 8	S	S
2368	Gravel	Siliceous	42 8	S	S
3229	Rock	Limestone	44 4	S	S
3805	"	"	46 9	Q	U
3725	"	"	49 2	Q	U
3816	"	"	50 5	Q	U
3015	"	"	54 3	U	U
2675	"	"	60 1	U	U
3669	"	"	62 0	U	U
2674	"	"	65 9	U	U

ords However, the attempt has been made to show a maximum value for the Los Angeles percentage of wear which includes all materials of satisfactory trend of the specification limits which have already been established for the Los Angeles test by several State Highway Departments

TABLE 8

COMPARISON BETWEEN LOS ANGELES TEST RESULT AND SERVICE RECORD BY WISCONSIN STATE HIGHWAY DEPARTMENT

Sample No	Type of material	Description	Los Angeles per cent of wear	Service record		
				Concrete	Bituminous construction	Traffic bound surfacing
1	Rock	Trap	11 1	S	S	S
4	Gravel	Igneous	18 4	S		
5	Rock	"	18 9			S
6	Gravel	Mixed igneous	19 4	S		S
7	"	" "	19 4	S		
8	"	" "	20 2	S		
9	Rock	Dolomitic	21 8			S
10	Gravel	Mixed igneous	22 2	S		
11	"	" "	22 3	S		
12	"	Igneous	22 3			S
13	"	"	22 5			S
14	"	"	23 5	S		
15	Rock	Dolomitic	25 1	S	S	S
16	Gravel	"	25 2			S
18	Rock	"	26 0		S	S
20	Gravel	"	27 3	S		S
21	"	Mixed igneous and sedimentary	27 7			S
22	Rock	Dolomitic	28 1	S	S	S
23	Gravel	"	28 2	S		S
24	Rock	"	28 4			S
25	Rock	"	28 4			S
27	Gravel	Mixed dolomitic and igneous	30 1	S		
28	"	Dolomitic	30 6	S		
29	Rock	"	30 8			S
30	Gravel	"	31 5	S		S
31	Rock	"	33 7	S		S
33	Gravel	Mixed dolomitic and igneous	34 1	S		
34	Rock		35 3		S	
35	Gravel	Dolomitic-igneous	35 5	S		
37	"	Mainly dolomitic	36 1			S
38	Rock	Dolomitic	36 3			S
39	"	"	38 0			S
41	Gravel	Mainly dolomitic	39 3	S		
43	Rock	Dolomitic	43 8			Q-U
46	"	"	47 2			S

service record and excludes the majority of materials which have proved unsatisfactory In general, it will be noted that the values given for bituminous surfacing are somewhat lower than those for concrete, and those for surface treatment are still lower This follows the general

COMPARISONS WITH OTHER TESTS

Figure 2 shows the results of tests made by the Michigan State College comparing the loss in the Los Angeles test with the loss in a circular track roller test In both tests all materials were prepared with the following grading.

Passing	$1\frac{1}{2}$-inch	sieve,	percent	100
"	$1\frac{1}{4}$-inch	"	"	80
"	1- "	"	"	60
"	$\frac{3}{4}$- "	"	"	40
"	$\frac{1}{2}$- "	"	"	0

The material passing the no 10 sieve after each test was considered as the loss and was expressed as a percentage of the original weight of the material In the roller test, the samples were subjected to a maximum of 500 passes of a cast iron roller loaded to 200 lb per inch of width These results show a definite correlation between the two tests It is interesting to note that 500 passes of the roller were required to secure a loss approaching that obtained in the Los Angeles test

Data comparing the loss in the Los Angeles test with flexural or compressive strengths of concrete have been furnished by several State highway departments, and are shown in Figures 3 to 6, inclusive In Figure 3 flexural test results representing a large number of tests made by the Georgia State Highway Department over a period of three years show a very good correlation with the results of the Los Angeles test Proportions specified by Georgia for concrete pavement con-

TABLE 9

Comparison Between Los Angeles Test Result and Service Record by Country Roads Board, State of Victoria, Australia

Sample No	Type of material	Description	Los Angeles per cent of wear	Service record
9534	Rock	Basalt	12 4	Excellent
9708	"	"	12 9	"
9416	"	Dolerite	15 3	Very good
9359	"	Rhyolite	15 5	Satisfactory
9483	"	Basalt	16 8	Very good
9419	"	Dacite	18 1	Satisfactory
9349	"	Basalt	22 0	Good
9704	"	Limestone	23 5	Fair
9351	"	Basalt	24 3	Good
9472	"	"	24 3	Fair to good
9550	"	Sandstone	28 6	Fair
9348	"	Basalt	28 8	Fair
9350	"	"	30 0	Fair to good
9464	"	Granite	34 6	Fair
9275	Gravel		35 0	Fair
9352	Rock		37 0	Poor
9499	"	Quartz	41 0	Fair (light traffic)
9573	"	Sandstone	43 6	" " "
9345	"	Quartzite	44 0	" " "
9265	"	Shale	46 0	" " "
9357	"	Sandstone	54 8	" " "
	"	Quartz	55 0	Poor

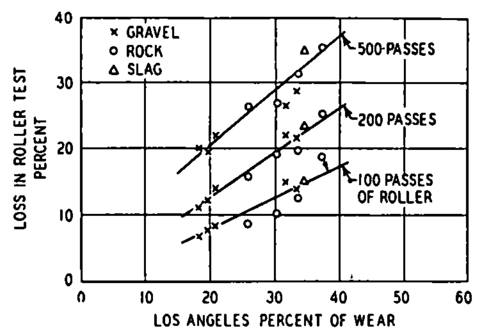


Figure 2 Relation Between Losses in Los Angeles Test and Circular Track (Roller) Test (Data from Michigan State College)

struction were used in this work Figure 4 presents the results of a laboratory investigation made in Texas with four different aggregates a hard limestone with a Los Angeles loss of 29 1 percent, a blue-gray quartzite with a loss of 33 1

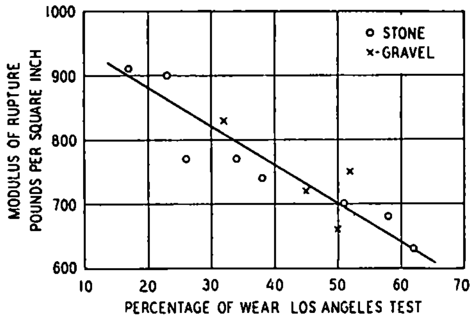


Figure 3 Relation Between Flexural Strength of Concrete and Los Angeles Abrasion Loss (Data from Georgia State Highway Department)

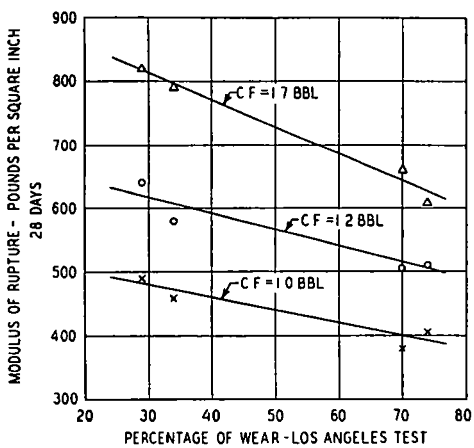


Figure 4 Relation Between Los Angeles Abrasion Loss and Flexural Strength of Concrete. (Data from Texas State Highway Department) .

percent, a red sandstone with a loss of 70 percent, and a soft limestone with a loss of 74 2 percent The materials were prepared to approximately the same grading, and made into concrete with three different mixes using 5, 7, and 9 gallons of water per sack of cement Due to differences in the aggregates, slightly different cement factors (sacks per cubic yard of concrete) were obtained for each water content The average cement factor for the 9-gallon mix was about 1 0 bbl , for the 7-gallon mix it was about 1 2 bbl and for the 5-gallon mix it was about 1 7 bbl To permit ready comparison with other data these average cement

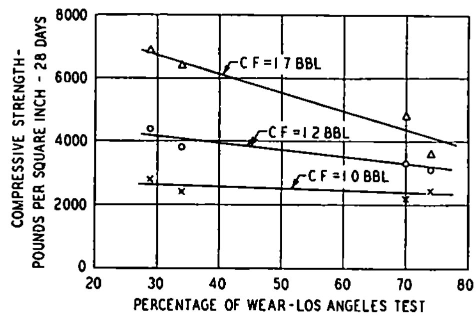


Figure 5 Relation Between Los Angeles Abrasion Loss and Compressive Strength of Concrete (Data from Texas State Highway Department)

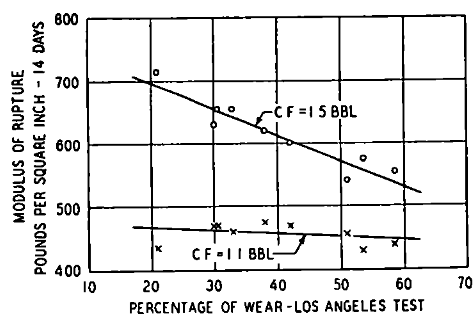


Figure 6 Relation Between Los Angeles Abrasion Loss and Flexural Strength of Concrete (Data for Crushed Stone from Nine Commercial Quarries in N Carolina)

factors have been indicated in the figures in place of the governing water content It will be noted that the effect of quality of aggregate is more pronounced in the rich mix than in either of the others This tendency is also revealed in Figure 5 which shows the corresponding relation between Los Angeles abrasion loss and crushing strength

Figure 6 gives the results of a series

of tests involving samples of granite and limestone from nine commercial quarries in North Carolina Similar relations to those indicated in Figure 4 are found These data emphasize the importance of taking into consideration the influence of the quality of the coarse aggregate in connection with the design of paving mixtures In the case of the nine commercial crushed stones in North Carolina a maximum difference of about 150 lb in modulus of rupture resulted when the cement content was held constant at 1 5 bbl Mixes designed for strength would compensate for this as well as other variables inherent in the method of arbitrary proportioning However it should be recognized that characteristics of aggregates other than resistance to abrasion such as, for example, surface texture, will also affect the strength of concrete

Data furnished by the Pennsylvania Department of Highways comparing the Los Angeles test result with determinations of the amount of soft or friable particles in gravel are given in Figure 7 In the test for soft pieces, particles between ⅝ in and 1¼ in were loaded without impact to a total of 200 lb, and larger particles to 400 lb Particles which crushed under these loads were considered to be soft or friable A definite relationship is found between the test for soft pieces and the Los Angeles test although several samples depart somewhat from the general trend

CONCLUSION

Some difficulty was found in attempting to summarize the data discussed in this paper This is due to the fact that all states do not have the same range in quality of coarse aggregrates, and one state with an abundance of hard rock will consider certain test values for quality as necessary which could not be used by other states Consequently it must be emphasized that recommendations given here are subject to change to suit local conditions

In general, it appears that there is a definite relation between the loss in the Los Angeles test and the service record of materials used in concrete, bituminous construction and surface treatment On the basis of the data available, the following percentages of wear appear to be

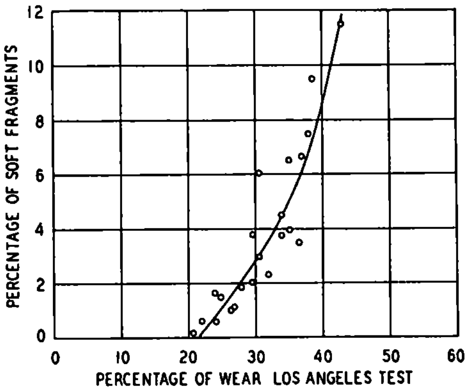


Figure 7 Relation Between Percentage of Soft Fragments in Gravel and Los Angeles Abrasion Loss (Data from Pennsylvania State Highway Department)

suitable for use in specifications to control the quality of coarse aggregates

	Percent
Concrete	50
Bituminous surfacing	40
Surface treatment	40

Definite correlations between the loss in the Los Angeles test and the strength of concrete are found, the lower the percentage of wear the higher the concrete strength Definite correlations are also found with the results of a circular track roller test and a test for soft or friable pieces

In conclusion, it appears that the Los Angeles test gives accurate indication of the quality of the material under test, and that its use in specifications controlling the acceptance of coarse aggregates is warranted

THE SIGNIFICANCE OF EARLY HEAT LIBERATION OF CEMENT PASTE

By R W Carlson

Associate Professor of Civil Engineering, Massachusetts Institute of Technology

SYNOPSIS

This paper concerns differences in rates of heat generation of cements during the first 24 hours after mixing, and the probable significance of the differences A "conduction" calorimeter is described that measures instantaneous rates of heat liberation The results from this calorimeter form the basis of the paper

Plotted curves of the rates of heat liberation of cements for the first 24 hours after mixing show wide differences, suggesting that the early heat characteristics may be a sensitive index to the performance of a cement The following properties of cement are shown to be related to early heat of hydration setting times, early strength, temperature rise of concrete in thin slabs, and probably bleeding of concrete

The early heat characteristics of cements are affected by the following factors fineness, composition, gypsum content, cooling treatment of cement clinker, temperature of hydration, "conditioning" on ageing of cement, and the presence of extraneous materials An example of the effect of these variables is the fact that a few per cent of an active puzzolanic material may increase the maximum rate of heat liberation by a hundred per cent or more, although it may reduce the total heat liberation

The preliminary studies of early-heat characteristics presented in this paper indicate that the conduction calorimeter is a good tool for research The early heat of hydration may provide a guide to improving the quality and uniformity of cements when the significance of results is more fully understood

The purpose of this paper is to present the preliminary results of studies on the early heat of hydration of cement The different heat characteristics of individual cements will be shown and the correlation of these characteristics with the performance of the cement will be attempted While it is believed that early-heat measurements have possibilities of worth while applications, the present paper is devoted only to indicating the apparent significance of the early-heat measurements The studies thus far are instructive but are not exhaustive enough to recommend early-heat measurement for purposes other than research

TEST METHODS AND APPARATUS

Because of the peculiar variations in early heat liberation of cement, three kinds of calorimeters were employed to obtain the complete heat-liberation record for 24 hours after mixing One simple calorimeter was used to determine the heat liberated up to one half hour, another gave the record from one half hour to about four hours, and a third gave the record up to 24 hours With the guidance of the test results obtained, one of these calorimeters may be omitted in future work

The first calorimeter, for immediate heat liberation, was simply a Dewar jar fitted with a sensitive mercury thermometer passing through a thick cork Cement was placed in the jar and cold water of known weight and temperature was added and mixed with the cement Temperatures of the paste were observed at 3, 15, and 30 minutes The heat liberation was computed by comparing these temperatures with the initial temperatures of the materials, taking account of respective weights and specific heats This method was not reliable beyond about 30 minutes because the heat transfer became large in comparison with the slow heat liberation at this time

The intermediate calorimeter was also a Dewar jar, but prepared to take a paste specimen sealed in a bottle In order to equalize the paste temperature, a copper rod extended through the cork to the bottom of the bottle An electrical resistance thermometer mounted on the copper permitted temperature measurement to 0 03° C Conversion of temperature rise into heat units necessitated knowing the heat capacity of all materials in the jar With this calorimeter, measurements could be taken even after the paste had hardened, without damaging the thermometer unit

The final, "conduction" calorimeter, to extend results to 24 hours, was of a new design, similar in principle to the vane calorimeter[1] It permitted the instantaneous rate of heat liberation to be determined at approximately constant temperature Heat was conducted away from a specimen almost as fast as it was liberated The rate of conduction was determined by observing the small temperature difference between the ends of a metal tube along which the heat was conducted This temperature difference could be measured readily to about 0 02° C with resistance thermometers and a test set designed to measure difference directly (If many tests are to be made, a recorder can be obtained that will make a continuous record on as many as 6 calorimeters simultaneously) A cross section of the conduction calorimeter is shown in Figure 1 The figure indicates how the heat is taken from the specimen by a copper rod, how the opposite end of the metal tube is maintained at substantially constant temperature, and how a Dewar jar prevents heat loss except in the direction of the metal tube Credit for suggesting this modification of the vane calorimeter is due I L Tyler, concrete technician for the Tennessee Valley Authority

The water cement ratio for most of the tests was 0 35 by weight This value made as dry a paste as could be stirred readily for the early tests It was found that the water content of the paste, within ordinary limits, had only a slight effect on the early heat liberation

The temperature of the hydrating paste was approximately 70° F In the conduction calorimeter, high-early-strength cements rose in temperature as much as 15° F during the period of rapid hydration, but the average temperature for 24 hours was not much above 70° F Proposed improvements in the calorimeter will reduce the variations in curing temperature by about one half

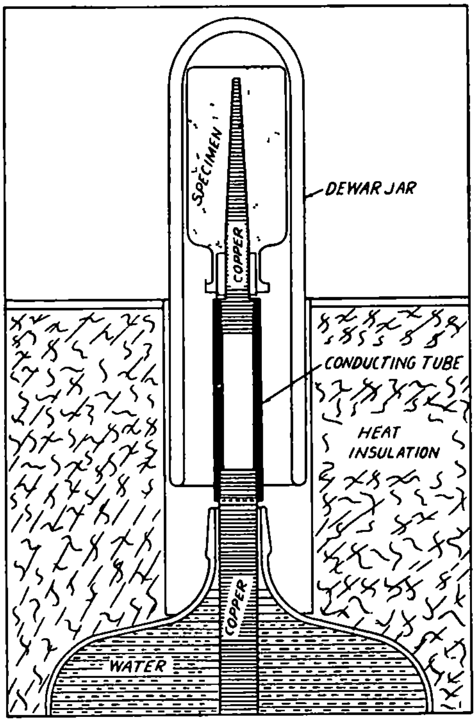


Figure 1 Conduction Calorimeter

[1] The Vane Calorimeter R W Carlson, *Proceedings* Am Soc T M 1933

GENERAL FINDINGS

Typical Heat Liberation of Hydrating Cement When water is first added to portland cement, there is an immediate heat liberation of appreciable amount

Only a small portion of the immediate heat is directly due to wetting the cement, the major portion is due to the dissolving of readily-soluble "impurities," such as free lime and probably alkalis. This heat continues to be liberated for a number of minutes at a decreasing rate until, at about two hours, it is imperceptible.

Before the "impurities" have ceased to liberate heat at a noticeable rate, the hydration of two of the major compounds, tricalcium aluminate and tricalcium silicate, begins to evolve heat at an increasing rate. Thus, a minimum rate of heat generation, at about two hours, is followed by a maximum rate at about 8 hours. The maximum is usually reached an hour or more after final set has been attained, and is followed by a gradual decrease in rate. At the end of 24 hours, the rate is only a fraction of the maximum, and if tests are continued for several days, the rate is found to be only about one per cent of the maximum.

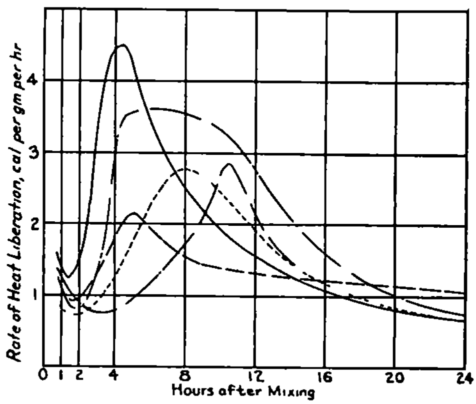


Figure 2 Early Heat Characteristics of Standard Cements

Many exploratory tests were made to determine what differences in early heat liberation might be expected due to various causes. Standard portland cements were found to vary widely among themselves. The differences were magnified by plotting curves of the instantaneous rates of heat liberation, rather than cumulative heat of hydration. A few such curves for selected standard cements are shown in Figure 2. Bearing in mind that these curves were all obtained on standard portland cements, the differences are striking. One need not conclude that standard cements are necessarily of wide variation in quality, but rather that the early-heat curves are a sensitive index of the early hydration of the cement.

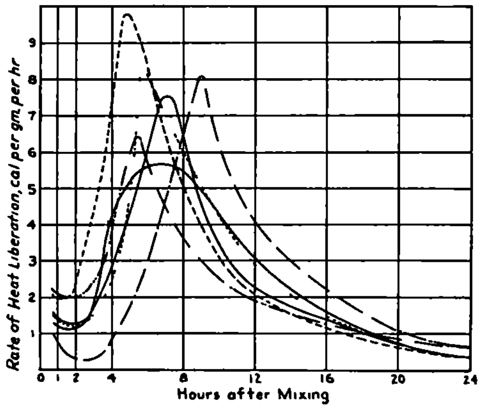


Figure 3 Early Heat Curves of High Early Strength Cements

Similarly, Figure 3 presents the early heat characteristics of selected high-early-strength cements. Note that the maximum rates of heat liberation are much greater than those of standard cements. Again, differences between cements are apparent.

Even greater differences are exhibited by different types of cement. In Figure 4 are shown selected heat curves for five types of cement: standard, high early strength, modified, low heat, and portland puzzolan. The particular cements selected were of nearly the same fineness, so the curves show in a general way the effect of composition. The standard cement was one of low tricalcium aluminate content, otherwise its curve would differ less from those of the two high-early-strength cements included The low-heat and modified cements represented in Figure 4 were closely similar in composition

Now that curves have been presented showing the different early-heat characteristics of cements, the questions arise as to what their significance may be and how the characteristics may be varied The effect of a few factors on the early heat will first be discussed, and later the correlation of early heats with properties of cements will be attempted

The variables that were found to have a pronounced effect on the early rates of heat liberation were

1 Fineness of cement,
2 Chemical composition,
3 Amount and condition of retarder,
4 Rapidity of cooling of cement clinker,
5 Temperature of hydration,
6 "Conditioning" or ageing of cement, and
7 Admixtures and impurities

Most of these variables have not yet been studied thoroughly, but some discussion of the effect of each on heat of hydration will be given before attempting the correlation of heat of hydration with performance

Effect of Cement Fineness The effect of finer grinding of cement is to increase both the maximum rate and the total amount of heat liberated during the first day of hydration The shape of the heat-rate curve is not greatly affected The general effect can be seen by comparing the curves of Figure 2 with those of Figure 3, because a single clinker was used in making each of several pairs of the particular standard and high-early-strength cements included

Effect of Composition Nearly all of the heat liberated between 15 minutes and 24 hours after the mixing of cement with water is believed to be due to the hydration of only two compounds, tricalcium aluminate and tricalcium silicate The remaining compounds are believed to contribute little to the early heat The heat liberation of dicalcium silicate, for example, attains a maximum rate of only about 0 1 cal per gram per hour during the first day, even when ground quite fine Of the two compounds largely responsible for the early heat liberation, tricalcium aluminate hydrates the more rapidly and is responsible for the abnormally high and abrupt heat liberation that is observed in some cements It seems safe to say that the majority of the tricalcium

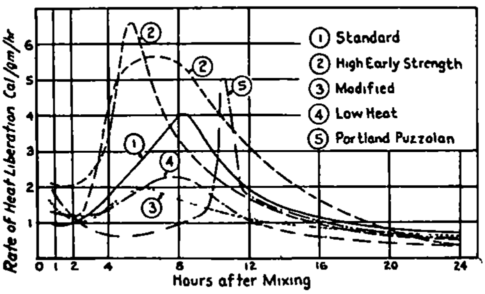


Figure 4 Early Heat Characteristics of Various Types of Cement

aluminate present in a fairly fine cement hydrates within 24 hours Since each one per cent of this compound has a potential heat liberation of 2 2 calories per gram of cement,[2] the proportion of the 24-hour heat attributable to tricalcium aluminate can be estimated Tricalcium silicate, being present in larger amount, may liberate more heat than the tricalcium aluminate, but at a more uniform rate

Effect of Gypsum A standard-cement clinker was ground with different amounts of gypsum to make five cements of equal fineness but with SO_3 contents ranging from 0 5 to 2 5 per cent The early heat curves for these cements are presented in Figure 5

[2] Woods, Starke, and Steinour, "Heat Evolved by Cement During Hardening" Eng News-Record, Oct 6, 1932

The cement with only 0.5 per cent of SO_3 exhibited a flash set with the liberation of a considerable amount of heat, following which there were several hours of slow hydration. The heat liberation then increased slowly and a low maximum was reached at about 15 hours.

With 1.0 per cent of SO_3, the rate of heat liberation during the first few hours was normal (as with 1.5 per cent SO_3) but a maximum was reached at the early age of 5 hours as compared with about 8 hours normal for this cement. The maximum rate was only about 60 per cent of normal but the rate at 24 hours

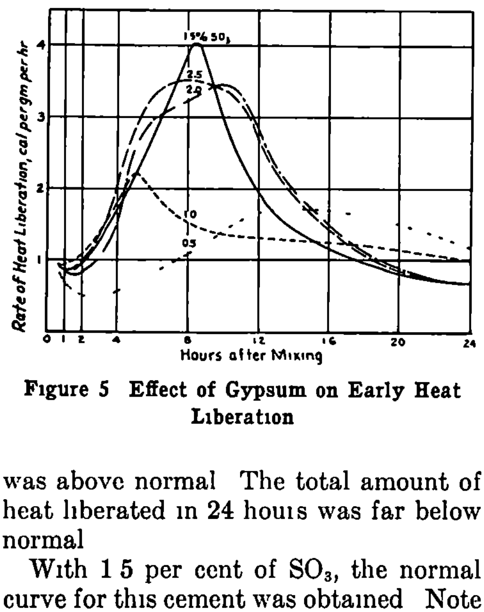


Figure 5 Effect of Gypsum on Early Heat Liberation

was above normal. The total amount of heat liberated in 24 hours was far below normal.

With 1.5 per cent of SO_3, the normal curve for this cement was obtained. Note that the heat-rate curve is sharp and that the maximum rate of heat liberation is higher than for any other amount of SO_3 investigated.

With 2.0 and 2.5 per cent of SO_3, the total heat liberation during the first 24 hours became progressively more extensive, but the greater liberation was attained through a prolonging of the period of rapid liberation and not by increasing the maximum rate.

The effect of different forms of gypsum, such as plaster of paris and the "anhydride" forms, have not yet been investigated. It would seem that the efficiency of the retarding action of various forms might be studied to advantage from the heat-of-hydration standpoint.

Effect of Rate of Cooling of Cement Clinker. Only a few tests were made with cements of different "glass" contents, such as result from different conditions of cooling of freshly-burned clinker. It is indicated, however, that while the "glass" content has little effect on cements containing small amounts of potential tricalcium aluminate, it has a large effect where the computed percentage of tricalcium aluminate is high. Due to the fact that preliminary tests were made on small specimens and therefore only qualitative, further tests are necessary before detailed comparisons can be presented.

Effect of Temperature of Hydration. The effect of raising the temperature of hydration was to increase both the maximum rate and the total amount of heat liberated up to 24 hours. The time required to reach a maximum rate of heat liberation was materially reduced by raising the temperature. An important effect of higher temperature appeared to be the reduced hydration at later ages, before 24 hours had elapsed, cements tested at elevated temperature were generating heat at a less rapid rate than were cements tested at normal temperature.

Effect of "Conditioning" of Cement. Two cements were treated with superheated steam so as to allow them to absorb a fraction of one per cent of moisture, according to the method developed and described by P. S. Roller[3]. It was found that the maximum rates of heat liberation were materially reduced by the conditioning, although the total heat liberation up to 24 hours was not greatly

[3] Roller, Paul S., "Seasoning of Cement at Elevated Temperature," *Industrial Engineering and Chemistry*, Vol. 28, March 1936.

affected Further studies are planned on conditioned cements

Effect of Admixtures and Impurities Admixtures of certain kinds affect the heat-rate curves of hydrating cements, while many have no noticeable effect one portland-puzzolan cement that is more or less typical Note the high, but very sharp, maximum rate of heat liberation that occurred at about 10 hours The puzzolanic material was responsible for the sharp peak on the heat-rate curve

TABLE 1

COMPARISON OF SETTING TIME AND EARLY HEAT OF HYDRATION FOR VARIOUS CEMENTS

Cement symbol[1]	Normal consistency	Specific surface[2]	24-hour heat of hydration[3]	24-hour tensile strength[4]	Time required for Initial set	Time required for 4 cal per gm[5]	Time required for Final set	10 cal per gm[6]
AH	27 0	2280	61	335	2 10	3 10	4 40	4 40
AS	24 5	1570	43	150	2 00	2 50	4 00	4 20
BH-a	27 0	2390	69	300	1 50	2 00	3 50	3 30
BH-b	26 0	2360	60	290	2 10	2 15	4 10	3 40
CS	24 0	1670	30	105	4 45	4 15	9 30	9 00
DH	25 0	2010	58	330	3 45	3 10	6 00	5 10
EH	26 0	1930	64	290	2 10	2 45	5 25	5 10
FH	26 5	2390	62	295	5 25	5 00	7 30	7 00
GH	26 0	2170	59	320	2 50	3 30	5 35	5 30
HH	25 0	2250	53	280	1 45	2 00	4 15	4 15
JS-a		1740	30			(Flash set)		
JS-b	24 5	1730	31	95	2 55	3 30	5 25	6 30
JS-c	24 0	1710	40	180	3 00	3 40	6 00	6 15
JS-d	24 0	1730	44	190	3 20	4 00	5 50	6 15
JS-e	24 0	1740	45	185	3 35	4 10	5 55	6 10
KP	29 0		26		3 45	3 30	8 45	10 00
LL	23 0		25		3 20	3 40	6 50	6 55
MM	23 0		26		3 50	3 40	6 35	6 50

Notes

[1] Second letter signifies type of cement (H, high-early strength, S, standard, P, puzzolan, L, low-heat, and M, modified)

[2] Square centimeters per gram as determined by Klein turbidimeter

[3] Calories per gram, exclusive of first ½ hour

[4] Pounds per square inch on standard mortar

[5] Exclusive of first ½ hour

[6] Exclusive of first ½ hour

Accelerating agents and retarding agents, including retarders other than gypsum, have especially large effects, while many common chemicals have little effect

Puzzolanic materials interground with cement altered the heat-rate curves, while the same materials simply mixed with cement had little effect Only puzzolanic materials that were reactive with lime, however, had a noticeable effect when interground with cement In Figure 4 there is included the curve of

RELATION BETWEEN HEAT CHARACTERISTICS AND PERFORMANCE OF CEMENT

Setting Times and Heat of Hydration After examining the cumulative heat curves of a number of cements, it appeared that initial set usually was reached when about 4 calories per gram had been liberated Similarly, final set usually corresponded to 10 calories per gram The immediate heat of hydration seemed to have no bearing on the setting times

In Table 1 are presented the measured setting times of a number of cements, and the corresponding times required for the heat of hydration to reach 4 and 10 calories per gram, respectively It may be noted that there is fair agreement between corresponding times In view of the fact that setting times are defined as

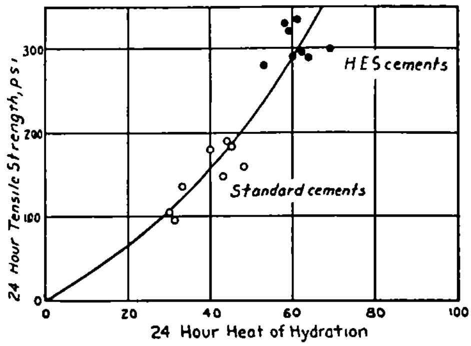


Figure 6 Relation Between Early Strength and Heat

the times required for arbitrary degrees of hardness and that they are not easily reproducible, the agreement is as good as could be expected Actually, the final set can probably be determined to a greater degree of reproducibility by heat measurements than by the usual method for determining final set The initial set is less definitely related to the heat measurement, because of the difficulty of separating immediate heat from the heat liberated by the hydration of cementing compounds

24-Hour Strength and Heat of Hydration The one-day tensile strength of standard mortar was compared with the one-day heat liberation for a number of cements from different plants A plot of the results is shown in Figure 6 and the data are also included in Table 1 It may be noted that the relation between heat liberation and strength is reasonably good, despite the fact that compositions and manufacturing methods were different The fact that the strength does not seem to be directly proportional to the heat liberation may be due to the somewhat different curing temperatures and water cement ratios of specimens for heat and strength tests, respectively

Strength results on the cements containing different amounts of gypsum are included in Table 1 Note that the increase in gypsum content has a large effect in increasing the strength and heat in the lower range (up to 1 5 per cent), but that further increase has little effect It is believed that, except for experimental error, the 2 5 per cent SO_3 should show slightly higher strength than 2 0 per cent because it shows more heat of hydration Here, again, the heat measurement is believed to be the more accurate means of determining relative values

Bleeding of Concrete and Heat of Hydration While no quantitative studies have yet been made on the relation between bleeding of concrete and heat of hydration, some connection has been observed In some cases, those cements that exhibited unusually low rates of heat evolution after the immediate heat had subsided, were observed to bleed freely An early gelation seems to be necessary to prevent bleeding No definite conclusions are warranted, however, on the connection between bleeding and early heat liberation, until test data are available

Temperature Rise of Concrete Slabs The opinion has often been expressed that temperature rise of thin concrete slabs due to heat of hydration is not a factor worthy of consideration The present discussion is not intended to magnify the importance of heat generation in thin slabs, but rather to evaluate the importance without distortion In view of the fact that pavements cast with rapid-hardening cements but protected from drying, have been observed to crack due to temperature drop well within 24 hours, the heat of hydration would seem to be a factor in summer concreting

The manner in which heat of hydration may cause cracking should be borne in mind While the temperature is rising, the concrete is soft and plastic When maximum temperature is reached the concrete is under almost no stress but has attained a fair degree of rigidity and has lost much of its ability to deform plastically As the temperature then drops, tension tends to develop in a slab of considerable length, because contraction is prevented If a temperature drop of 15 or 20 degree occurs quickly, the concrete may crack

An important fact is that even though a greater temperature is attained during subsequent hot seasons than is attained during early hardening, the maximum temperature during early hardening is the reference temperature Drying shrinkage and temperature drop from this reference temperature determine the net tendency of the slab to contract

Without forgetting that seasonal variations in temperature and drying shrinkage are likely to be larger factors, a specific example of the effect of heat generation is illustrated by the curves of Figure 7 The curves show the computed temperatures, for each of three classes of cement, in 10-inch slabs with one face insulated and the other maintained at uniform temperature A cement content of 1 5 bbl per cu yd of concrete was assumed The curves are therefore only applicable to this single set of assumed conditions and should not be construed as being general

The curves of Figure 7 show that in a 10-inch slab containing rapid-hardening cement a substantial temperature rise of over 30° F may occur Not every high-early strength cement would produce such a high temperature rise, a cement that was above average in abruptness of early heat liberation was selected for illustration On the other hand, still higher temperatures would be exhibited by a few cements The temperature rises shown for modified and low heat cements are

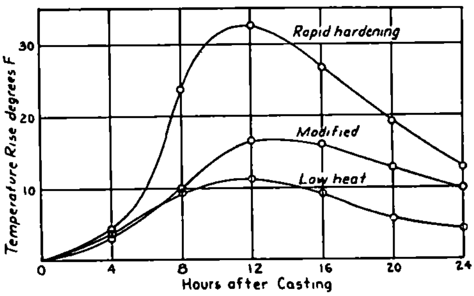


Figure 7 Effect of Cement on Temperature Rise in 10-In Slab

small enough to justify the conclusion that when they are used, heat generation is likely to be a small factor

CONCLUDING REMARKS

The test results and this discussion must be considered as preliminary and therefore no conclusions are drawn Cement research employing calorimetric methods will be continued in the hope of having more accurate and useful results to report in the future It is only hoped that this paper may help to give a better understanding of the early hydration of cement

A STUDY OF TRANSLUCENT ASPHALTIC FILMS

By J R Benson

Bituminous Engineer, Kansas Highway Commission

SYNOPSIS

Thin films of asphaltic materials have been weathered naturally and artificially and then studied microscopically by means of transmitted light One reaction, designated as coagulation, occurs in some positive Oliensis spot test materials under nearly all test conditions Observations and solubility characteristics indicate the coagulated film, at its formation time, to be composed of a dispersed solid phase and a continuous liquid phase This reaction is believed to be very important in that it gives a method of determining the compatibility of an asphalt mixture

Other characteristics such as hardening, checking, wrinkling, flocculation, precipitation, and the formation of waxy bodies have been observed microscopically Photomicrographs of these asphaltic film characteristics are presented

The determination of the quality and durability of bituminous materials is a perplexing problem of great economic importance

Low cost roads remain low in cost only so long as the elements of construction remain in serviceable condition While other elements enter, the bitumen remains a large factor in determining the success or failure of any bituminous construction

Lagging far behind the rapid development of the use of bituminous materials has been the development of accurate tests to determine the true quality and serviceability of these materials Many tests have been proposed to determine these factors, which are derived largely from the examination of successful materials through their physical characteristics The industry, of necessity, produces material to meet the conditions of test With changes of methods, the relationship between the test and quality and serviceability changes, and the determination of quality tends to return to its former indefinite status

This paper represents the work undertaken during the past two years to obtain a more satisfactory test for quality and durability of asphaltic materials A new method of attack has been used in a study of the reactions occurring in very thin translucent film of bitumen when exposed to agencies tending to decompose or alter the structure of the material

An examination of the character of decomposed material existing in bituminous mat surfaces in 1933 first directed attention to the possibilities of a microscopic investigation of asphaltic films A preliminary survey was begun in 1933 and was continued at intermittent intervals until a more complete investigation was started in the Road Materials Laboratory of the Kansas Highway Commission in June, 1936

Films of such thickness as to be translucent to light under the microscope permit a direct observation of the physical reactions taking place within the film This thinness permits acceleration of decomposition and alteration much greater than that obtainable in films of greater thickness

The reaction of translucent films has been studied under six conditions of exposure, classified as follows 325° F, 140° F, mild ultra-violet infra-red, cold quartz 140° F, natural weathering and by carbon dioxide and oxygen gases at 325° F The two tests designated by temperature were performed in darkness with the film in contact with air

The mild ultra-violet infra-red test was performed by use of a 250 watt

General Electric CX lamp. This lamp is a high temperature incandescent type with a special envelope. The characteristics as furnished by the manufacturer may be summarized as follows:

	Per cent
Ultra-violet, 3,500 to 4,000 A.U.	0.15
Visible, 4,000 to 7,500 A.U.	17.51
Infra-red, 7,500 to 26,000 A.U.	82.34

The cold quartz ultra-violet lamp uses a combination of mercury vapor and rare gases to produce radiations essentially those of mercury. The lamp normally operates at a temperature of approximately 115° F. The radiation data supplied by the manufacturer for the distribution between 1849 and 4358 Angstrom units may be summarized as follows:

	Per cent
2,536 A.U.	77.97
2,967 to 3,128 A.U.	11.69
3,657 to 4,358 A.U.	8.57

METHOD OF PRODUCING THE FILMS

It was found by experiment that film thicknesses of approximately 0.001 in. gave the most satisfactory results. This film is obtained by the use of a gage constructed of stainless steel which is drawn across the face of the slide on which is a small portion of the material to be tested (Figure 1).

Fluxed and liquid asphaltic materials require no heating to produce the desired film. Penetration or non-liquid asphalts require heating of the slide, asphaltic material and gage to the lowest temperature at which the film can be formed.

Film thicknesses greater than 0.001 in. have poor light transmission and render definition of the reactions difficult. Thinner films do not always produce reactions with sufficient clearness.

METHODS OF TEST

Heat: The heat tests were performed in the dark in constant temperature ovens with the film in contact with air. The first tests were run at a temperature of 325° F. This temperature represented a degree often reached in some paving operations and it was believed to be the maximum that could be used under the conditions of test.

Tests run at this temperature presented, in some materials, the greatest acceleration, reactions becoming visible in periods as short as one-half hour. Those materials reacting at this temperature usually gave strong indications of the reactions in 5 hr. and were well developed or completed in 24 hr.

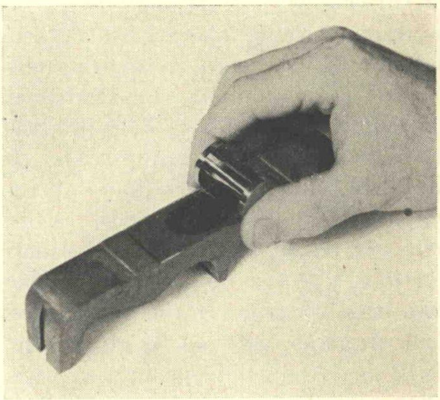


Figure 1. Gage for Forming Film

The second test was at 140° F. It was believed that this represented the maximum commonly reached by pavements in service, and that reactions presenting themselves at this temperature might be expected to give similar results in service with similar exposure.

The development of reactions at this temperature is slow in comparison with the time necessary at the 325° F. test. Tests at this temperature have been carried on over periods up to 2040 hr. Films presenting reactions in from two to five hours at 325° may require from 200 to more than 500 hours to present the reaction at the lower temperature. Materials have been found which present a reaction at one temperature and not at the other The reason for this is not definitely known

Infra-Red, Mild Ultra-Violet, 140° to 170° Test A General Electric CX Therapeutic lamp of 250 watt rating was used as a convenient method of obtaining mild ultra-violet radiations The large heat and visible radiation output of this lamp made temperature control difficult with the apparatus available It was not possible to hold the temperature at 140° as desired, but temperatures rose to 170°, which probably tended to accelerate reactions beyond that due to the ultra-violet radiations present

Cold-Quartz Ultra-Violet Test The fused quartz lamp, operating at approximately 115° F, uses as the radiation generating medium mercury vapor and a combination of gases which produce a strong radiation at 2536 Angstroms Initial tests run with this lamp at room temperatures of 70° to 90° F over a period of 60 days gave little distinctive reactions This lamp produces copious quantities of ozone In the initial experiment, the lamp was placed at a distance of 12 in from the specimens, with a cellophane shield placed 4 in above the tray The presence of large quantities of ozone above the shield may have effectively absorbed much of the ultra-violet present

A second investigation in which the specimens were placed 1½ in from the generating tube, fully exposed to the ozone present and maintained at a temperature of 140°, developed accelerated reactions in many materials, some of which presented no reactions at the other test conditions

Weathering Specimens were exposed to sunlight and outdoor air temperatures in a celluloid covered wooden box One group of specimens was run during the winter months, with a second series run during the summer months The temperatures in the box reached a maximum of 185° F, dropping to between 50° and 90° F during the night in summer exposure The winter exposures varied from about 70° to 10° F,

Gases and Heat Gas experiments were conducted at 325° F Oxygen and carbon dioxide were used, the gases entering one end of the sealed metal box and leaving at the other, controlled by a bubble flask on the outlet tube

The reactions as observed have been classified as follows A Clear, B Coagulated, C Flocculated, D Waxy bodies, E Checked, F Hardened The presence of scums, pits and wrinkling, when formed, has also been noted

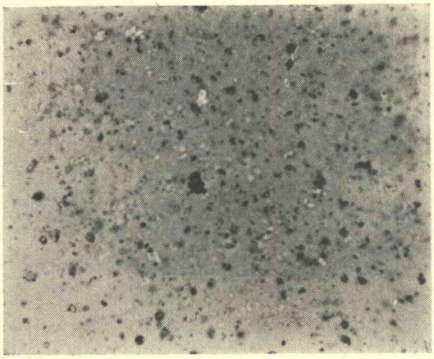
Of these reactions, coagulation is the most interesting in the forms in which it occurs, in the consistency with which it appears in certain types of material, and in the observations which have been made on its behavior This paper is therefore concerned largely with this reaction

Coagulation may be considered as the curdling or drawing together into nodular stringy form a part or the whole of the film The coagulation may consist of either a coagulated scum on the surface of the film, a single coarse structure, or it may be a lacy structure of great fineness composing the entire film

The formation of coagulation does not appear to be dependent on the presence or absence of the particles of carbon, sand, dirt or waxy bodies

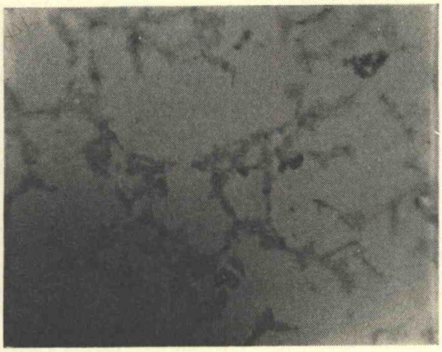
OBSERVATION OF THE COAGULATIVE REACTION

Development of the Reaction at 325° F The development of coagulation in a positive Oliensis SC3 material is shown by progressive photomicrographs at 430X in Figure 2 The new specimen has a considerable quantity of suspended flocculent and carbonaceous material These materials have partially dissolved and partially coalesced at the end of 30

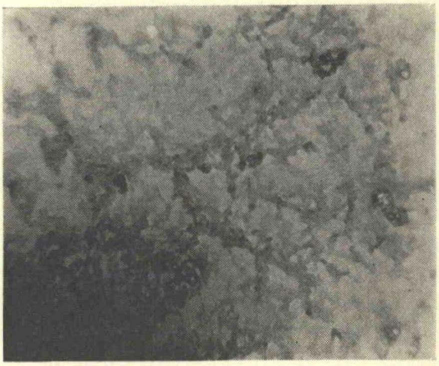


New Sample

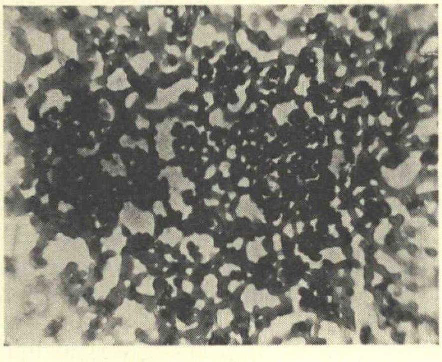
30 Minutes



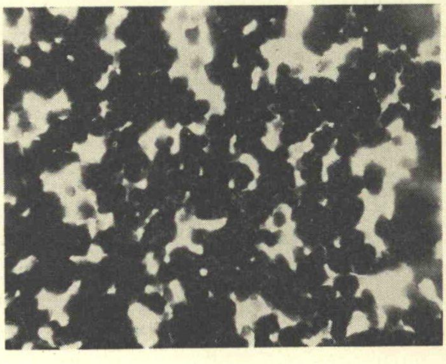
90 Minutes



135 Minutes



255 Minutes



12 Hours

Figure 2

min at 325° F The beginning of the coagulative reaction is also visible in the formation of numerous small light areas

Coagulation has begun to assume definite form at the end of 90 min of heating by the appearance of stringy dark matter At the end of 135 min this material has increased both in quantity and in size Portions of the coagulated material appear to form within the body of the film

A large portion of the film is in coagulated form at the end of 255 min The structure has become coarse A thin film of material remains on the surface of the slide This film is of a light yellow color and careful observation reveals that this film also presents a very fine coagulated structure

The reaction is complete at the end of 12 hr with the material in a very coarse, nearly opaque structure

Coagulation as a Separation Into Two Distinct Phases It is believed that coagulation is caused by a separation of the asphaltic material into two distinct phases One phase, that producing the actual stringy or lacy structure, appears to separate from the body of the material, and is of more consistency than the original or the remaining material The separation of this phase causes the remaining material to become more fluid and lighter in color

The second phase, the light yellow fluid, is not so evident in the tests run at high temperatures as it is in those run at lower temperatures and especially under the cold-quartz-140° test Coagulative specimens under the 325° temperature, if observed during the formation of the coagulation, will usually show clearly the separation of the solid phase The second phase appears to decrease after formation is in an advanced stage, appearing as a light colored layer of thin material on the surface of the slide Careful observation will usually show this material also to have assumed a coagulative form The decrease in the quantity of the liquid phase may be due partly to a change into coagulated material and partly to evaporation or to some other reaction at high temperature

Coagulative specimens at low temperatures (140° F) and natural weathering may show considerable separation into a solid and a liquid phase The presence of two phases is observable in the lacy or granular structure obtained in some coagulative materials under the cold-quartz-140° test as in Figure 3 Observing the grooving of the film under the microscope, the structure is evident throughout the film with the solid phase as a very fine, delicate material

When coagulated films of cutback asphalts are disturbed with a needle, the liquid phase quickly flows into the cracks, and more slowly into the groove produced by the needle The observations, made at room temperature, indicate a liquid phase of very low viscosity

Many coagulated films produced under a temperature of 140°, natural weathering or the quartz lamp exhibit a two phase system consisting of a continuous fluid phase and a discontinuous solid phase The solid phase appears to be composed of numerous small round or oval shapes which stick together and give the appearance of a lacy structure The fluid phase fills the interstices, but on prolonged exposure, tends to collect in an exceedingly thin layer between the surface of the slide and the film of granular coagulated material

Effect of Solvents on Coagulated Films The coagulative reaction during the development period is reversible by the addition of such solvents as CS_2, CCl_4, or benzol Both phases are soluble in these solvents during the development stages, but become insoluble or difficultly soluble after prolonged treatment The action of the solvent is to recombine the phases, or to dissolve the coagulated matter On evaporation of the solvent, the film is smooth and clear, with little or no evidence that the material had previously been in the coagulated state When subjected to the test conditions the film again develops coagulation

Effect of Gases The tests with various gases at 325° F are given in Table 1

It is evident that oxygen may cause a slight acceleration while CO_2 causes considerable negative acceleration, or possibly prevents coagulation, as in samples 96 and 97, when the formation is compared to air Some samples have been observed which have coagulated when heated, protected by a cover glass and therefore not in contact with air It appears that air and oxygen accelerate, but are not essential for coagulation of some materials This may be important in asphalt-aggregate structures where coagulation of the asphaltic material may occur without the presence of air

TABLE 1

Sample No	Oxygen—O_2		Carbon dioxide—CO_2			
	2 hours	4 hours	2 hours	4 hours	6 hours	11 hours
92	Clear	Clear	Clear	Clear	Clear	Clear
93	Clear	Clear	Clear	Clear	Clear	Clear
94	Clear	Clear	Clear	Clear	Clear	Clear
95	Clear	Clear	Clear	Clear	Clear	Clear
96	Coag	Coag	Clear	Clear	Clear	Clear
97	Coag	Coag	Clear	Clear	Clear	Clear
98	Coag	Coag	Clear	?	Coag	Coag
71	Clear	Clear	Clear	Clear	Clear	Clear
66	Clear	Clear	Clear	Clear	Clear	Clear
70	Clear	Clear	Clear	Clear	Clear	Clear
73	Coag	Coag	Clear	Clear	Clear	Coag
74	Clear	Clear	Clear	Clear	Clear	Clear

MATERIALS INVESTIGATED

The materials investigated may be classified as follows

Group 1 Seventeen samples, composed of RC, MC and SC type materials, produced at seven refineries and representing asphaltic material used in Kansas during 1936 for bituminous mat construction Both positive and negative Oliensis materials are represented

Group 2 Eleven samples of special material to investigate methods of manufacture and the effect on the reactions

Group 3 Fourteen samples, consisting of two series of tests made up of the originals and blends of the originals to produce both negative and positive Oliensis material

In addition to these groups, more than 150 samples of material, including original samples of asphaltic oils used since 1932 and 43 samples of asphalt extracted from existing pavements and mats, have been investigated by two or more of the methods given in this paper

Group 1

This group (Table 2) is composed of nine negative Oliensis and eight positive Oliensis spot test materials The Kansas specifications for positive Oliensis materials contained a special low-penetration ductility clause as follows the SC type of material and the distillation residues of the cutback type materials MC and RC shall be reduced to the designated penetration for that type by the method A S T M D243-28T The ductilities at the given penetration shall be 50 cm + at 77° F, 5 cm per minute The materials shall be reduced as follows

R C Cutback	40-50 penetration
M C Cutback	25-35 penetration
S C Asphaltic oil	15-25 penetration

Results All of the nine negative Oliensis materials remained clear at the end of the 325° and 140° heat tests With one exception, all of the negative material remained clear at the end of the CX, cold-quartz and natural weathering tests One sample, No 71, coagulated at the end of 142 hr under the CX lamp, at the end of 97 hr under the cold-quartz lamp and was coagulated at the end of 1176 hr of natural weathering

The positive Oliensis material presents more variation in behavior Five samples presented coagulation on all tests, three, Nos 65, 66 and 71, gave clear reactions at the end of the 325° test, two of these, Nos 65 and 71, gave clear reactions at the end of 2040 hr at 140° F No 65 also remained clear at the end of 1176 hr of natural weathering

It is therefore apparent that some positive material will show considerable resistance to coagulation It is interesting to note that the three positive Oliensis materials which show the greatest resistance to coagulation are of the SC type

Group 2

Samples Nos 77, 78, 79 and 80 are penetration asphalts produced from Venezuelan crude Two samples, 77 and 78, are negative Oliensis materials of 150 and 10 penetration respectively The others, 79 and 80, also of 150 and 10 penetration, are positive Olienses materials produced at a different refinery The difference in behavior of these materials with respect to either the Oliensis or the penetration test is not great The one difference is exhibited in sample No 80, the 10 penetration positive Oliensis material which had coagulated at the end of 1792 hr at 140° F

Samples 92 to 98 are asphaltic ma-

TABLE 2

GROUP 1

No	Type	Source	Spot	325°		140°		G E-C X 140°-170°		Cold quartz 140°		Weather	
				Clear	Coag	Clear	Coag	Clear	Coag	Clear	Coag	Clear	Coag
				Hours	*Hours*	*Hours*	*Hours*	*Hours*	*Hours*	*Hours*	*Hours*	*Hours*	*Hours*
59	MC2	K-1	N	278		2,040		250		250		1,176	.
60	RC2	K-1	N	278		2,040		250		250		1,176	.
61	SC5	K-1	N	278		2,040		250		250		1,176	
62	MC2	K-2	P		18		528		67		17		192
63	RC2	K-2	P		18		528		84		17		192
64	SC3	K-2	P		18		528		67		17		192
65	SC3	T-1	P	278		2,040			250		17	1,176	
66	SC3	Mo-1	P	278			648		190		17		75
67	MC2	Mo-1	N	278		2,040		250		250		1,176	
68	MC2	T-2	N	278		2,040		250		250		1,176	
69	RC2	T-2	N	278		2,040		250		250		1,176	
70	SC3	O-1	P	278		2,040			310		35		27
71	SC5	T-2	N	278		2,040			142		97		1,176
72	MC2	K-3	P		18		528		67		17		192
73	RC2	K-3	P		2		528		48		17		24
74	MC2	K-3	N	278		2,040		482		149		1,176	
75	RC2	K-3	N	278		2,040		250		250		1,176	

terials produced by different processes. Samples 92, 93 and 94 are vacuum processed asphalts which have been progressively blown. No. 95 is a mixture of blown vacuum processed and blown fire-distilled materials. No. 96 is a cracked unblown asphalt; No. 97 is the same material blown. No. 98 is a mixture of a cracked and a blown fire-distilled asphalt. No. 92, the straight negative Oliensis material, resisted coagulation in all tests, as did also No. 93. No. 94, more highly blown than No. 93, showed less resistance and had coagulated at the end of 149 hr. under the cold-quartz-140° test. No. 95 indicated still more susceptibility to coagulation, and had coagulated at the end of 149 hr. under the cold-quartz-140° test and also at the end of 238 hr. under the CX lamp.

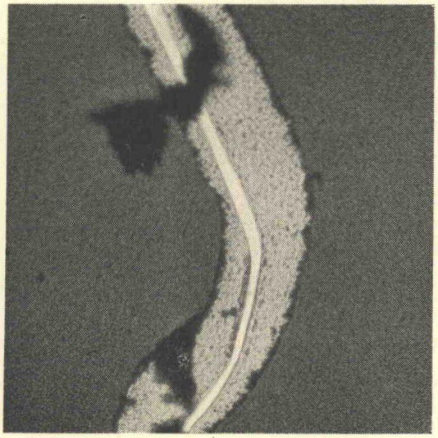


Figure 3. Lacy Coagulated Structure. 200 X

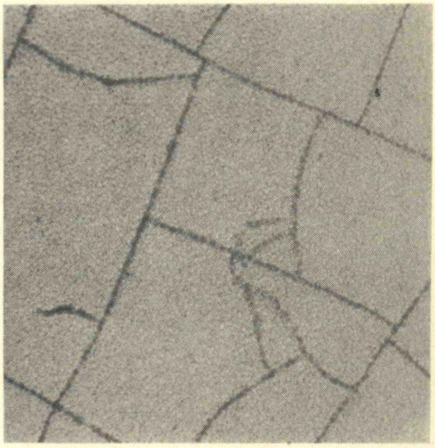


Figure 4. Coagulated Unblended Asphalt. 430 X

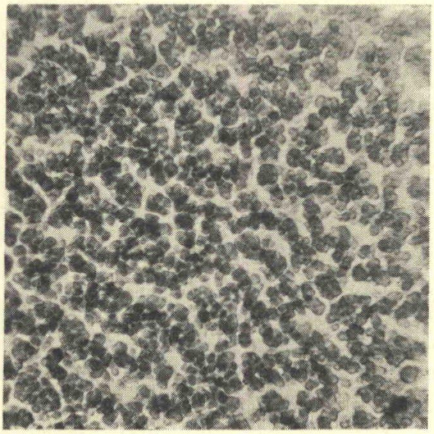


Figure 5. Coagulated Blended Asphalt. 430 X

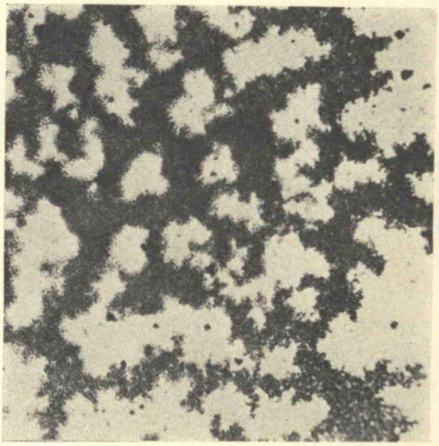


Figure 6. Flocculation and Coagulation. 430 X

Samples 96, 97 and 98 coagulated under all tests. The effect of blowing and blending is noted in the very rapid coagulation obtained in samples 97 and 98, where coagulation had taken place at the end of one-half hour at 325°. An acceleration may also be noted in the time required for coagulation under the CX lamp and weathering

Group 3

Results "A" Series This series presents coagulation beginning with sample A4, although the spot is still negative All samples containing more positive material than No A4 present coagulation at the end of the test

The condition noted in sample No 5B, which did not present the coagulative reaction at the end of 1052 hr at 140°, was checked by additional samples It appears that an equilibrium of blending occurred at this point It was noted that the spot from this sample appeared less positive than those immediately preceding, with smaller quantities of positive material in the blend It was first believed that an error had occurred in

TABLE 3

GROUP 2

No	Type	Source	Spot	325°		140°		G E -C X 140°-170°		Cold quartz 140°		Weather	
				Clear	Coag	Clear	Coag	Clear	Coag	Clear	Coag	Clear	Coag
				Hours	*Hours*	*Hours*	*Hours*	*Hours*	*Hours*	*Hours*	*Hours*	*Hours*	*Hours*
77	150 Pen	Venz	N	118		1,792		482		149		268	
78	10 Pen	Venz	N	118		1,792		482		149		268	
79	150 Pen	Venz	P	118		1,792		482		149		268	
80	10 Pen	Venz	P	118			1,792	482		149		268	
92	Vac proc	Not blown	N	118		1,152		482		149		268	
93	Vac proc	Blown slightly	N	118		1,152		482		149		268	
94	Vac proc	Medium blown	N	118		1,152		482			149	268	
94+		Blown fire-distilled	P	118		1,152			238		149	268	
96	Cracked	Unblown	P		4		72		94		17		75
97	Cracked	Blown	P		½		72		46		17		24
98	Cracked +	Blown fire-distilled	P		½		72		46		17		24

"B" Series The "B" series tests were extended to include 1052 hr at 140° F The asphaltic materials are from a different source from those used in the series "A" tests In this series, series "B," two samples, 4 2B and 4 5B, presented coagulation at the end of 16 hr at 325° although the spot remained negative The 325° test was consistent, giving coagulative reactions on all samples after 4B

Coagulation did not develop in the 140° test until the spot had changed to positive in sample 4 7B The long exposure necessary to develop the reaction at this temperature is to be noted

the quantities involved, but additional samples of the same quantities corroborated the original results No coagulation resulted on this sample at the end of test, but a dark scum had formed on the surface

The results of this test make evident the differences in the behavior of materials when subjected to the two tests of 325° and 140° It will be noted that other materials have at times given differences in the other direction, being clear at the end of the 325° test and coagulated at the end of the 140° test

The effect of blending on the character of the coagulation is of interest as shown in Figures 4 and 5 Figure 4 is a photomicrograph of sample 9AB, the straight positive material Coagulation is of a fine grain and the material is of a light yellow color (32 hr at 325°)

Contrast Figure 4 with Figure 5, exposed the same period under the same conditions This material, sample 8B, has presented a much coarser coagulation than the original positive material This condition has been noted in other blended materials and points to such blending as one source of severe coagulative reactions

Wrinkling The coagulative reaction must be differentiated from that of wrinkling (Figure 10) The latter reaction often appears in films presenting no coagulative reaction, particularly under low temperature tests The wrinkling reaction is distinguished by the lack of any evidence of a separation into different constituents such as occurs in the coagulative reaction Wrinkling appears to be caused by the formation of a tough scum on the surface of the film, which, due to subsequent shrinkage of the material below the scum, develops a wrinkled appearance Films which are wrinkled, but show no evidence of coagulation, are designated as clear

TABLE 4

DATA, SERIES A, GROUP 3

Specimen	Positive %	Negative %	Spot	325° 32 hours
A0	0 00	100 00	Neg	Clear
A2	9 00	91 00	Neg	Clear
A3	13 00	87 00	Neg	Clear
A4	16 00	84 00	Neg	Coag
A5	18 00	82 00	Pos	Coag
A5 5	19 00	81 00	Pos	Coag
A6	20 00	80 00	Pos	Coag
A6 5	21 50	78 50	Pos	Coag
A7	23 00	77 00	Pos	Coag
A8	26 00	74 00	Pos	Coag
A10	100 00	0 00	Pos	Coag

TABLE 5

DATA, SERIES B, GROUP 3

Specimen	Positive %	Negative %	Spot	325° 16 hours	325° 116 hours	140° 836 hours	140° 1,052 hours
1AB	0 00	100 00	Neg	Clear	Clear	Clear	Clear
1B	6 48	93 52	Neg	Clear	Clear	Clear	Clear
2B	9 00	91 00	Neg	Clear	Clear	Clear	Clear
3B	12 02	87 98	Neg	Clear	Clear	Clear	Clear
4B	15 00	85 00	Neg	Clear	Clear	Clear	Clear
4 2B	16 00	84 00	Neg	Coag	Coag	Clear	Clear
4 5B	17 00	83 00	Neg	Coag	Coag	Clear	Clear
4 7B	17 50	82 50	Pos	Coag	Coag	Clear	Coag
4 8B	17 75	82 25	Pos	Coag	Coag	Clear	Coag
5B	18 00	82 00	Pos	Coag	Coag	Clear	Scum
6B	20 05	79 95	Pos	Coag	Coag	Clear	Coag
7B	22 53	77 47	Pos	Coag	Coag	Clear	Coag
8B	25 00	75 00	Pos	Coag	Coag	Coag	Coag
9AB	100 00	0 00	Pos	Coag	Coag	Coag	Coag

Pits: Clear films may also contain numerous pits of the type shown in Figure 9. The cause of the pitting, which has been observed in several films, is not known. When this reaction is present, careful observation and an extension of the test is often necessary to determine whether the reaction is pitting, waxy bodies or the beginning of a coagulative reaction.

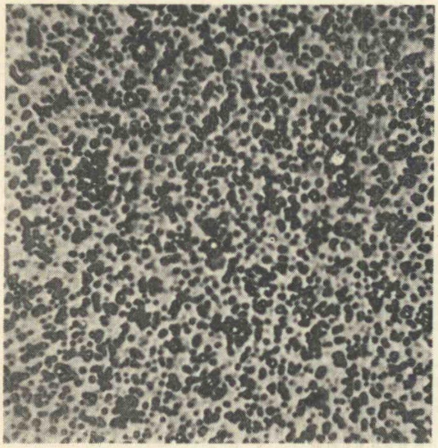


Figure 7. One Form of Coagulation. 430 X

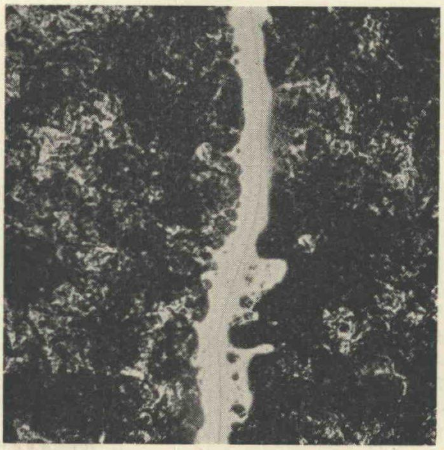


Figure 8. A Grooved Coagulation Film. 200 X

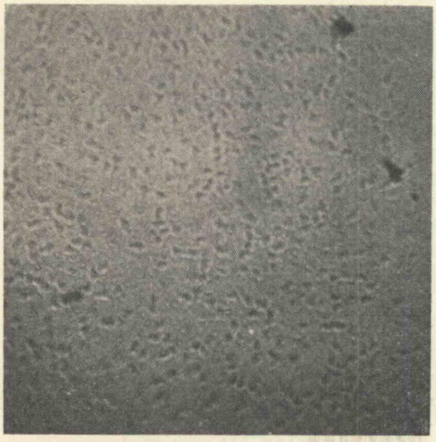


Figure 9. Clear Film with Pits. 200 X

Figure 10. Wrinkled Clear Film. 200 X

OTHER REACTIONS DESCRIBED

Flocculation: This material has been observed and photographed by other investigators, one of the latest by Lewis and Hillman,[1] the term "carbonaceous flecks" being applied. Due to the reversible solubility of this material, it appears that the material approaches an asphaltene in character.

The flock appears more commonly in positive Oliensis asphalts, particularly fluxed materials having high percentages of light distillates, such as the RC2. An increase in the amount of flock present has been noted in material of this type, RC2, which has been stored over a period

[1] *Public Roads*, Vol. 18, No. 5, July 1937.

of several months. When coagulation takes place in the presence of the flock, the flock is incorporated in the coagulative reaction. This is evident in Figure 6.

Waxy Bodies: This reaction was one of the first to be observed in the original investigation. The appearance of this material is evidently due to the crystallization of the waxy material into various forms and sizes. The waxy material is apparently of less gravity than the surrounding asphaltic material and forms on the surface of the film. The reaction is best observed under a cover glass, the crystals collecting on the under side of the glass in a fairly uniform layer. The crystals are optically active under polarized light, as shown in Figure 11, taken by this means.

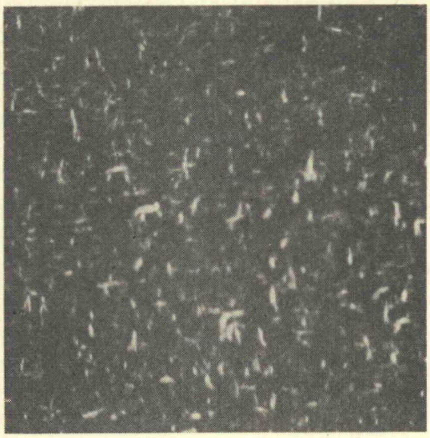
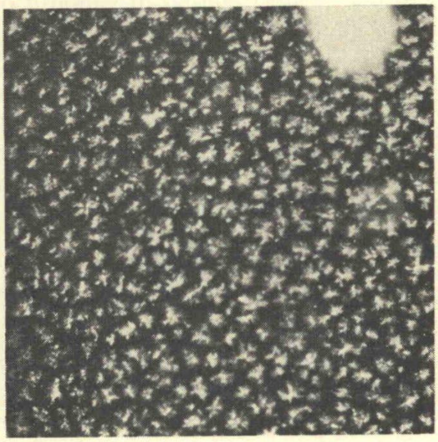


Figure 11. Wax by Polarized Light. 200 X

Figure 12. Wax Crystals. 200 X



200 X

430 X

Figure 13. Crystalline Wax Form at Two Magnifications

Waxy material seems to be present in negative Oliensis materials to a greater extent than in positive Oliensis asphalts. Some negative Oliensis materials give considerable amounts of the crystals, as is evident in Figure 12. This material had a ductility of 100 cm. + on the penetration distillation residue It is apparent that the presence of the material is not indicated to a large extent by the ductility of the material It also appears possible that the amount of crystal precipitated is not closely related to the amount of waxy material present, but to the solubility of the waxy material in the asphaltic medium Other forms of the waxy bodies after crystallization are shown in Figure 13

The material may be crystallized out by heating to temperatures as low as 140° for a short period and then cooling to room temperatures The crystals do not immediately appear on cooling, but will show halos or circles of lighter color than the surrounding film after from 10 to 20 min, and form crystals after perhaps 30 min The formation is slow

Checking This reaction consists of the development of checks and cracks which usually extend through the thickness of the film The reaction is not regarded as having any relationship to the coagulative reaction Checking usually occurs as straight or smoothly curved lines which divide the film up into comparatively large geometrical figures Checking may occur in either clear or coagulated films Checking in a coagulated film is shown in Figure 4, checking in clear films is shown in Figure 14

The most severe checking appears in naturally weathered specimens, particularly those exposed during winter months, where all the specimens developed severe checking Temperature change and film brittleness appear to be the major factors causing checking

Similar reactions have been used as a determination of failure by some investigators [2] The importance of the reaction has not been determined in this investigation

Hardening As to the relative rates of hardening, a number of materials have been investigated by observing the condition of the film when grooved at room temperatures by a sharpened needle, the operation being performed under the microscope at 200 X Several degrees of plasticity may be noted in the soft films, the consistency varying from sticky and adhesive to gummy and dead The ultimate point of brittleness may be observed by the chipping and cracking of the film when grooved The appearance of a plastic and a brittle film is shown in Figures 15 and 16 respectively

[2] Strieter, *Proceedings,* A S T M, July 1936

SUMMARY

The study of translucent asphaltic films as presented in this paper is intended more as a survey of the reactions and the possibilities of the methods as a medium for further research, than as an intensive investigation into the factors involved

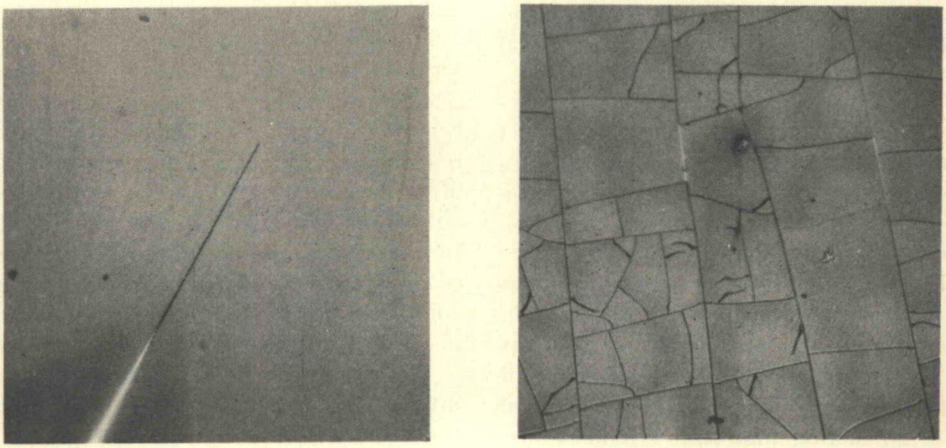
It is believed that the coagulative reaction is a form of actual decomposition of the asphaltic structure, and as such, its observation has possibilities as an accelerated weathering test The test also appears indicative of incompatible blends of asphaltic materials, and may be of assistance in securing more efficient blending operations in production

Correlation with actual performance of coagulative susceptible and coagulative resistant materials, particularly with regard to the positive Oliensis materials, is not complete It is known that some coagulative susceptible materials have given poor service in road construction, particularly in penetration type treatments

Tests performed on SC type positive Oliensis asphaltic materials extracted from bituminous mats which have given excellent service for periods of from six to seven years, have shown considerable resistance to the coagulative reaction Materials extracted from two sheet asphalt pavements in service 20 and 26 years, respectively, with excellent service records, were also highly resistant to coagulation.

Several materials of the SC positive Oliensis type in which both the original and extracted materials were highly susceptible to coagulation, have given fair service in dense crushed limestone bituminous mats for periods up to four years, with sealing operations performed during the last two years. More research is indicated to establish definite correlation between the reaction and actual serviceability.

The reaction of solvents in reversing the coagulative reaction may be important in the analysis of coagulative materials extracted from bituminous mats and pavements. The refluxing of the material by solvents may give an extracted material which would bear little relationship to the material existing in the aggregate-asphalt structure with the bitumen in a coagulated state.



430 X

200 X

Figure 14. Checking in Clear Films

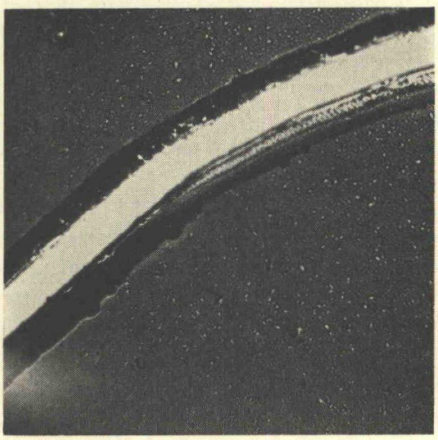


Figure 15. Plastic Film. 200 X

Figure 16. Brittle Film. 200 X

The separation of the asphalt material into two phases, one with a very low viscosity, increases the opportunity for preferential adsorption into certain aggregates such as limestone The material remaining as a binder would have few characteristics of the original material, and perhaps of the extracted material if the second phase were also extracted

The formation of the coagulative reaction in darkness and out of contact with air, at elevated temperatures, indicates the possibility of a similar reaction occurring in pavement structures with those materials

Flocculation of some materials appears to be due to solubility equilibriums The formation of the flocculent material may be either accelerated or reversed by the addition of carbon tetrachloride It has been noted that materials having appreciable quantities of flocculent matter usually are susceptible to coagulation

The effect of the presence of waxy bodies has not yet been determined Several materials which possessed good ductility character when subjected to regular laboratory tests, have been noted to have poor adhesive qualities when used in construction These materials, when investigated in translucent films, presented considerable quantities of waxy bodies Figure 12 is a photomicrograph of one of these materials

From theoretical considerations, the presence of this material appears objectionable The waxy material, being of less specific gravity (from 31° to 34° API)[3] than the asphaltic medium, tends to accumulate on a surface above the film From observations of this formation, it appears possible that actual displacement of asphaltic material may occur in favor of the waxy material The ultimate development of a waxy layer of material effecting more or less complete separation of the body and the asphaltic material is, from this view point, quite possible

This condition would result in a loss of bond between the constituents of an asphalt-aggregate structure with resultant failure, with the cause of failure not evident in any standard adhesive or ductility test The behavior of this material is being investigated

Checking of films is apparently accelerated by film brittleness and rapid temperature change Very brittle films often do not check if cooled slowly, but will check if cooled quickly The reaction might be used to indicate brittleness of the film under constant rate of cooling conditions The value of the reaction has not been determined in this investigation

The rate of hardening of the asphaltic material may be determined by observing the time required for reduction to a brittle film It is believed that this method may be of value in determining more accurately the true rate of hardening of asphaltic material when used in service, conditions approaching the test condition This condition is approached closely in asphaltic material used for sealing or surface treatments, where thin films are exposed to the action of the elements

There does not appear, at the present time, to be very much correlation between the rate of hardening as observed in the film investigation, and that secured from the material in service when incorporated in an asphalt-aggregate structure It has been noted that some extracted materials from pavements and bituminous surfaces which have given excellent service, and are still in excellent condition, have very high rates of hardening when in thin films at elevated temperatures It is evident that the aggregate structure is protective to a high degree, in the prevention of volatilization of the asphaltic material The very fine material present in some types of asphalt-aggregate structures, particularly with

[3] Nelson, W L , *Petroleum Refinery Engineer*, McGraw-Hill, p 581 (1936)

fluid or semi-fluid asphalts, appears to be of as much, or perhaps more importance in the protection of the asphaltic material than in the mechanical stability which this material gives to the completed structure

CONCLUSIONS ON THE COAGULATIVE REACTION

1 The coagulative reaction, as observed in some forms, is a separation of the asphaltic material into two phases, one tending towards hardness, the other towards liquefaction

2 The coagulative reaction may be produced in some materials by heat in darkness and out of contact with air

3 Coagulation may be produced by exposure to sunlight

4 Coagulation is accelerated by contact with air and oxygen and retarded by carbon dioxide

5 Coagulation is accelerated by exposure to ultra-violet radiations

6 Coagulation occurs more frequently in positive Oliensis materials than in negative

7 Some positive Oliensis materials have a high resistance to coagulation

8 Certain types of blending increase the tendency to coagulate

9 Coagulation may be produced at temperatures normally reached by pavements in service

10 The coagulative reaction is reversible at certain stages by the addition of solvents

ACKNOWLEDGMENTS

The writer wishes to acknowledge the work of Mr C J Becker, who, as assistant during the past few months, has contributed greatly to the knowledge of the behavior of asphaltic films as presented in this paper, and also the valuable constructive criticism on the coagulative reaction which has been given by Mr L M Law, Mr G L Oliensis and Dr Hans Winterkorn Appreciation is expressed for the special materials furnished by Mr Law and Mr Oliensis

Grateful appreciation is expressed for the encouragement and assistance given the investigation of asphaltic films during the past four years by Professor C H Scholer of Kansas State College and by Mr Frank S Gilmore of the Asphalt Institute

REPORT OF COMMITTEE ON MAINTENANCE COSTS

By H K Bishop, *Chairman*
Chief, Division of Construction, U S Bureau of Public Roads

SYNOPSIS

The project consists of compiling and classifying maintenance costs for the purpose of establishing a fair basis for comparison among the various States Through the facilities of the U S Bureau of Public Roads, 47 States have cooperated in a uniform study of the cost of maintaining various types of surfaces, comprising a total of 18,716 miles Reports are now tabulated for a three-year period for 30 States, a two-year period for 11 States, and a one-year period for 6 States Variations in costs were found to be pronounced Assuming proper design and construction, factors largely responsible for this variation are listed as budget limitations, differences in cost of labor and materials, traffic, climate, quality of maintenance and width of pavement Because figures over so short a period are apt to be misleading, the Subcommittee will continue to accumulate these costs over a period of years

The problem of the Subcommittee is to present comparable highway maintenance costs with carefully defined contents The principal value of such data among agencies charged with highway maintenance, and material men interested in the economy of products, lies in their being available for comparison on a fair basis. Maintenance costs must, therefore, be accumulated over a period of years as averages and weighted to a common base for quality of maintenance, difference in labor, material and equipment prices and other factors affecting the data, such as character and weight of traffic, classification of maintenance operations, deferred maintenance work, inadequate design, drought, wind erosion, frost action, poor subsoil conditions, inadequate drainage, slides and floods

The work of the Subcommittee started in 1933 with a survey, through the facilities of the U S Bureau of Public Roads, of maintenance costs available at State highway departments, the principal agencies charged with highway maintenance It was found that the meaning and contents of the maintenance dollar had as many variations as there are States In order to secure uniformity it was proposed that each State select about twenty representative sections, each section composed of one type of pavement and cross section, and report the cost on a form having a uniform classification of maintenance operations and other pertinent data necessary to give uniform meaning and interpretation to the maintenance costs Forty-seven States expressed a desire to cooperate in the study and 1,233 sections were selected, comprising 18,716 miles of highways These sections are representative of the various types of surfaces now in use

We now have reports under these standards for a three-year period from 30 States, for a two-year period from 11 States and for a one-year period from 6 States These reports were reviewed and tabulated during the past year In some instances they were returned with suggestions for revisions when not prepared with the uniformity desired Correspondence and conferences were had with respect to omissions and revisions in reports and clarification of abnormal costs At the same time the comparative weight of the various conditions affecting maintenance costs were studied Maintenance engineers of the U S Bureau of Public Roads also made inspections and reports with respect to the quality of maintenance on the sections for correlation with the costs It is felt that this work focused attention on the need for uniform classification of maintenance operations and definition of nomenclature and is bringing about a better understanding by maintenance men of the distribution and meaning of costs

The period for which the information has been collected is too short to give representative figures for the different highway surfaces Reference to our cost summary sheets at this stage of the study, however, indicates that there are large variations in cost per mile for projects of the same type Also there are considerable variations in the detail items making up the total cost per mile on projects of the same type Traffic density does not appear to have a consistent effect on the costs, probably due to the fact that other conditions overshadow the effect of volume of traffic

Assuming a proper design in the original construction including drainage, subgrade, base support and thickness of surface, there are several factors probably responsible for the large variation of costs for similar types, mentioned in order of their importance, as follows

1 Budget limitations
2 Variations in labor, material, equipment and overhead costs
3 Traffic, amount and weight
4 Differences in climate, including drought, moisture, temperature and snowfall
5 Quality of maintenance and efficiency of maintenance organization
6 Width of pavement and nature of shoulder materials

In order to illustrate the difficulties encountered in attempting to make comparisons of costs between States and types of surface, we will quote some of the data obtained from the study to date

LABOR COSTS

Let us first take the question of labor costs in six of the States which reported maintenance costs for a two-year period Table 1 gives some interesting information

The average rate for common labor in the first group is 45 and 23 cents per hour, respectively, in the second group 50 and 40 cents, and in the third group 55 and 30 cents Apparently labor rates have a very important effect on the maintenance costs It is, however, also evident that labor rates are not entirely responsible for the widespread differences in costs between the three groups of States in this tabulation Traffic has a decided influence, the quality of maintenance has an influence and, of course, climate and available funds must be taken into consideration

TABLE 1

HIGH TYPE PAVEMENT

Group	Location of State	Maximum traffic		Average annual maintenance cost per mile
		Cars	Trucks	
1	Central State	6,160	757	$355 36
	South Atlantic State	816	349	116 02
2	Pacific Coast State	4,077	780	640 15
	Central State	1,548	143	150 87
3	North Central State	2,845	427	240 86
	North Central State	801	149	104 51

TRAFFIC

Now let us take the question of traffic A study of Table 1 gives some indication of the influence of this factor on maintenance costs Take for instance the first group of two States where the maximum traffic is 6,160 cars and 757 trucks against

816 cars and 349 trucks We have an annual maintenance cost per mile of $355 36 against $116 02 In the second group where the maximum traffic is 4,077 cars and 780 trucks against 1,548 cars and 143 trucks, we have a maintenance cost per mile of $640 15 against $150 87 In the third group of States where the maximum traffic is 2,845 cars and 427 trucks, the maintenance cost per mile is $240 86 against $104 51 It appears from these data that traffic has a direct influence on maintenance costs Some indication of the influence of weight of traffic may also be had from observing that in the States with high maintenance costs, truck traffic is about two to four times the truck traffic in the States with low maintenance costs

The comparison just made refers to one type of pavement Let us examine other types and observe whether a similar traffic influence is apparent Comparing a high type of pavement in a New England State on which the maximum traffic is 8,349 cars and 604 trucks against a Central State with a traffic of 598 cars and 51 trucks we have an annual maintenance cost per mile of $329 65 against $154 26 On a medium type pavement in a Western State having a maximum traffic of 1,941 cars and 216 trucks against a South Atlantic State with a traffic of 606 cars and 255 trucks we have a maintenance cost per mile of $131 60 against $334 34 On a low type pavement in a North Central State with a maximum traffic of 1,299 cars and 153 trucks against a Central State with 387 cars and 166 trucks the maintenance cost per mile is $355 12 against $951 72 The last two groups of States show inconsistencies as far as traffic is concerned

The high maintenance costs on the low traffic roads just cited are undoubtedly due to other factors which can be described only after a more extensive analysis of the data submitted Weight must be given to the fact that in some States considerable roadside maintenance work having no direct relation to traffic is included in the maintenance costs It is also necessary to consider that some of the high traffic roads with high costs are located in States having severe winter climatic conditions requiring expenditures for snow removal, ice treatment and other traffic service charges included in the maintenance costs These may alter an apparent relation between traffic and maintenance costs Therefore in the final analysis of expenditures by types of surface the cost of the pavement and shoulders will have to be segregated from the total expenditures for all maintenance operations Regardless, however, of the inconsistencies shown in some of the cases cited it is felt that the remaining maintenance costs presented and other cases not referred to herein for which data are available justify a conclusion that traffic, particularly weight, does have an important influence on maintenance costs

In order to bring out more forcibly the difficulty involved in a study of this nature and the variations that occur let us take the yearly costs in the same State In a Central State the weighted average maintenance cost per mile of all sections having a high type of pavement was $363 07 per mile in 1934 while the cost per mile on the 23 sections included within the average figure ranged from $213 41 to $1,017 72 per mile In 1935 the weighted average cost per mile in the same State was $347 66 while the range was from $215 74 to $1,094 44 per mile On one section of high type pavement in a Western State the cost per mile varied from $513 19 in 1934 to $1,197 25 in 1935 In another Western State the cost per mile on a section also having a high type of pavement varied from $341 37 in 1934 to $1,045 00 in 1935

When routine maintenance of any element considered in this study has been more or less neglected for a year, or perhaps two years, it usually follows that the deferred maintenance in the succeeding year shows a cost that would seem out of all proportion, while at the same time, if the job is well done in the third year, there may be only slight charges against the same element for a year or two following To illustrate this point we cite a section of highway in a Southern State No expenditure for drainage appears to have been made on this section in 1937 while is was necessary in 1936 to expend $212 05 for cleaning out, widening and lowering ditches in addition to extensive repairs to drains which amounted to $388 02 The reason for the heavy expenditure in this case was due to the fact that the particular element had been neglected for some time prior to 1936 These variations in cost illustrate the importance of analyzing maintenance data over a long period of years and the danger of drawing conclusions from such information gathered over a short term of years

WIDTH OF SURFACE

Now let us take the width of surface and see what influence it has on maintenance costs We have a Western State with 18-ft pavements and an average cost per mile of $348 00, three Central States, one with 18-ft pavements and an average cost per mile of $363 00, a second State with 18-ft pavements and a cost of $333 00 per mile, a third State with 20-ft pavements and a cost of $155 00 per mile, two North Central States, one with 20-ft pavements and $189 00 per mile and the second State with 20-ft pavements and $344 00 per mile, and one Pacific Coast State with 20-ft pavements and an average cost of $706 00 per mile The above figures refer to a high type of surface It may be well to note that the high maintenance cost in the latter State is largely due to extensive shoulder patching The shoulder maintenance cost on one section with a 20-ft pavement included in the average was $777 33 per mile, practically all expended for patching The traffic on this particular road was 2,991 cars and 835 trucks We might conclude from the information reported that this extensive shoulder repair was due to heavy traffic, particularly trucks getting off the pavement However, we take another section in this same State with a traffic of 5,254 cars and 1,219 trucks and we find that the shoulder maintenance is only $66 53 per mile—again showing inconsistency

Let us examine a little more into the subject of width of pavement. Let us take a North Atlantic State which has a considerable mileage of 16, 18 and 20-ft pavement

From Table 2 it will be noted that the cost for maintaining a 16-ft pavement is $341 82 per mile, the cost for maintaining an 18-ft pavement is $398 02 while the cost of a 20-ft pavement is $592 25 Considered on the basis of width alone this must mean that the greater the width of pavement the higher the maintenance cost, which is certainly erroneous

The figures again show that the 16-ft pavement has an average traffic of 999 vehicles per 24 hours while the 18-ft pavement has an average traffic of 1,269 vehicles for 24 hours This is in approximate proportion to the cost On the 20-ft pavement we have an average traffic of 3,043 vehicles per 24 hours which is about three times the traffic on the 16-ft width and 2½ times the traffic on the 18-ft width, while the cost of maintenance for the 20-ft pavement is about 1 7 times the cost for the 16-ft width and about 1½ times for the 18-ft width Undoubtedly the 20-ft width being of later construction and on the heavier traffic roads received more intensive maintenance than the 16 and 18-ft widths located on the lesser important route From the information we now have there is still a question of the extent of the influence of width of pavement on maintenance costs

While the Subcommittee is able to report progress and that a large amount of available data are being accumulated, the period of time has been too short to present information that would be of value in answering the question before us. To give out at this time information relative to comparative maintenance costs would be misleading and of no particular value. The Subcommittee, therefore, must ask for further time for study of this complicated and perplexing question.

TABLE 2

NORTH ATLANTIC STATE

HIGH TYPE PAVEMENT

Year	Width	Mileage	Cost per mile	Traffic count		
				Average	Highest	Lowest
1935	16	22.76	$237.28	945	1,350	450
	18	98.64	304.25	1,274	3,600	621
	20	50.48	465.25	2,200	4,100	398
1936	16	13.09	467.55	1,016	1,350	620
	18	92.59	529.30	1,269	3,950	417
	20	50.55	729.94	4,133	8,973	1,819
1937	16	13.09	397.86	1,075	1,350	750
	18	78.10	360.80	1,262	4,345	650
	20	41.80	579.12	2,744	4,895	1,050
3-year weighted average	16	16.31	341.82	999	1,350	576
	18	89.78	398.02	1,269	3,936	559
	20	47.61	592.25	3,043	6,057	1,092

DISCUSSION ON MAINTENANCE COSTS

CHAIRMAN DICKINSON. One element which is very important and which was not covered in the report is the question of inventory. We must know what we have left over at the end of the year. For instance in determining maintenance costs of an automobile it does not do much good to keep track of what is spent unless we can also determine the real value of the vehicle at the beginning and end of the year. There is no good way of determining the current value of the road which we have had at the beginning and again at the end of the year and yet it seems that is an important element in the determination of current cost. A store has to make an inventory once a year of what it has on the shelves or else it cannot tell very much where it does stand.

MR BISHOP. We realize that the question of depreciation has to be taken into consideration. In the sections chosen we took pavements in good condition and with good foundations and of practically the same age in each case, and still the cost figures barely reflect the cost of maintaining that pavement in its original condition during its life. We realize that at the end of that life it must be replaced but the great trouble at the present time is that the pavements have to be replaced before they are worn out because they are obsolete. While we build pavements to last 10, 15 or 25 years, by changing the automobile types which last two or three years, we are running 1937 automobiles over 1922 or earlier constructed pavements.

REPORT OF DEPARTMENT OF TRAFFIC

C J TILDEN, *Chairman*

REPORT OF COMMITTEE ON TRAFFIC REGULATION IN MUNICIPALITIES

ONE-WAY STREETS

By W S Canning, *Chairman*
Engineering Director, Keystone Automobile Club

SYNOPSIS

The purpose of this study is to lay before public officials the reasons for and the effects of the designation of existing two-way streets for the use of traffic in one direction A municipality may provide for increased traffic volume in this manner at practically no cost Other reasons for one-way streets may be reduction of accidents, provision for heavy directional flows of traffic at certain hours, or the control of progression at desirable speed One-way streets practically eliminate head-on collisions and side-swipe accidents between vehicles proceeding in opposite directions It restricts the necessary scope of attention for both operators and pedestrians, reduces probability of turning accidents and eliminates headlight glare

Among the disadvantages arising from one-way street traffic is the necessity for additional travel distance to reach certain destinations, the increased accident exposure to vehicles and pedestrians by reason of traversing two or more additional intersections, and the probability of accidents and confusion where streets are not one-way throughout

A "one-way street" may be defined as one upon which all vehicular traffic flows in the same direction at the same time within the entire width available for such traffic

For the purpose of this investigation the scope is limited to those streets that were originally designed and used for two-way traffic but later restricted to movement in one direction

In general, a one-way street may be thought of as one-half of a two-way street Obviously, the traffic stream in the opposite direction, displaced by the restriction to one direction, must be provided for on some immediately adjacent street It follows that an adjacent street, or streets, must be available to accommodate the displaced flow, which shall have sufficient unused capacity at the time the demand is made to absorb the traffic displaced from the first street, or failing that, shall be designated for one-way traffic in opposite direction from the other street

A combination of two one-way streets, though separated by a city block, should have effects upon accident hazards and facility of movement similar in many respects to those of the rural highway upon which opposing traffic is separated by divisional islands Modifying elements in this direct comparison, are the probable presence of parked vehicles on city streets and reduced sight distances at intersections as compared to divided highways However the same general effect on accidents should follow even though the surrounding conditions are quite different

REASONS FOR ONE-WAY STREETS

1 To provide for increased volume of traffic without prohibiting parking by segregating opposing streams

2 To afford immediate relief to traffic congestion at substantially no cost

3 To facilitate movement on service streets or alleys, on which parking is of primary consideration, and which may be too narrow to accommodate two-way traffic

4 To discourage the use of narrow or residential streets by burden vehicles or thru traffic to by-pass a signalized intersection

5 To conform to street car movement

6 To reduce the accident hazard

7 To meet demand for heavy directional flows of traffic at certain hours, or under certain conditions

8 To provide means of controlling progression at desirable speed

9 To facilitate or simplify signalization of a complicated intersection where one or more streets may be of minor importance or little used

Other Advantages are

1 Practical elimination of head-on collisions

2 Elimination of the discomfort of glare at night and the accidents caused thereby

3 Practical elimination of sideswipe collisions between vehicles proceeding in opposite directions

4 Probable reduction of sideswipe collisions with parked vehicles

5 Restriction of scope of attention of operator on the one-way street and for operators and pedestrians crossing it

6 Reduction of hazards to pedestrians crossing the one-way street, both at intersections and between intersections

7 Reduction of probability of turning collisions

8 Better control of progressive signalization at speeds approaching theoretical capacity

9 More nearly complete use of street area through alternation of directional flow

Some of the Disadvantages to be Considered are

1 The necessity for additional travel distance to reach certain destinations

2 Additional accident exposure by reason of traversing two or more additional intersections to (a) vehicular traffic, and (b) pedestrians

3 Unless a street is designated for one-way traffic throughout, there is probability of accident and confusion at the termination of the two-way movement

FACTUAL DATA

Little factual data to support the opinions advanced appears to be available

Philadelphia Figures Seem to Indicate

1 That speeds have been materially increased

	M P H
Chestnut Street when two-way (1929)	
Westbound	17 6
Eastbound	17 9
Eastbound (1928)	17 3
Since made one-way	
Eastbound (1937)	21 1
An increase of 20-22 percent	

	M P H
Walnut Street when two-way (1929)	
Westbound	16 8
Eastbound	18 6
Since made one-way (1937)	
Westbound	24 8

2 That traffic volumes carried have increased entirely out of proportion to the increase in general traffic

7 a m 7 p m	1929 (two-way)	1934 (one way)	Per cent increase
Chestnut at 52nd	7,137	12,938	81.5
Walnut at 52nd	4,052	12,098	200

Motor vehicle registrations in Philadelphia and adjacent Delaware County in same years were

1929	322,685
1934	328,358
Increase 1 75 percent	

Gasoline taxed in Pennsylvania in same years were

1929	900,495,620
1934	1,136,343,197
Increase 26 percent	

3 That accidents between intersections on one-way streets are materially reduced

	At inter sections	Between inter sections
Chestnut Street (One-way)	108	10
Walnut Street (One-way)	162	7
Market Street (Two-way)	302	82
South & Spruce (Two-way)	284	52

OPEN QUESTIONS

1 Width of cartway

2 Effect upon (a) Business areas, (b) residential areas

3 Objections advanced by the public

DISCUSSION ON ONE-WAY STREETS

CHAIRMAN MORRISON Perhaps some of the conclusions that Mr Canning has not been able to get from other sources would be available from those in the audience This report is open for discussion

MR M O ELDRIDGE, *Department of Vehicles and Traffic, District of Columbia* We have tried rush hour one-way operation on four different streets in Washington and in that way we have in some cases doubled the traffic, and I believe in one case almost quadrupled it On Sixth Street we not only operate on a rush hour one-way basis but the traffic lights have been arranged with triple offsets so as to give a true progression so that when one enters that street and moves through at 22 to 23 miles per hour, one can move from one end of the street to the other without stopping

We are now trying an experiment with neon lights The neon light is arranged with an arrow which is turned on by a clock at 4 o'clock and remains in operation, indicating the direction in which traffic should move until 6 o'clock Then the arrow goes off In addition to the arrow the letters "one-way" are shown and the arrow flashes, indicating clearly to those who approach the intersection that this is one-way operation and the direction in which the traffic should go I know that we have had considerable reduction in accidents on those streets

Very recently one-way operation has been developed through the driveways in the Parks also with greatly increased volume of traffic

The big advantage in rush-hour one-way operation over parallel one way streets is that everybody seems to be in favor of it and the drivers themselves help to enforce it When they meet a man going in the wrong direction they indicate very clearly that he is on a one-way street and so we have no difficulty at all in enforcing the rule We are increasing our volume, increasing our speed and reducing our accidents by this means

MR CANNING Recently in Philadelphia we had plenty of experience with a directional one-way street You might have heard about the Army and Navy game there We made Broad Street and most of the north and south streets in Philadelphia directional one-way south-ward becoming northward after the game There were some 5,000 cars more this year than last year and I am told that approximately 9,000 vehicles per hour were handled on Broad Street during the rush hour southbound

CHAIRMAN MORRISON Mr Canning, do I understand that all of the traffic moved in that direction at the same time?

MR CANNING All traffic moved in the same direction at the same time

PERCEPTION AND VISIBILITY OF AUTOMOBILE LICENSE PLATES

By Milton H Aldrich

Assistant Professor of Civil Engineering
University of Vermont

SYNOPSIS

Increasing motor vehicle speeds and interstate travel have emphasized the need for easy and accurate identification of vehicles under various traffic conditions, while growing registrations have tended to cause crowding of license plates and the use of legends difficult to perceive and retain Tests have accordingly been devised to compare the merits of plain numeral and letter-numeral systems, various sizes and styles of legends, and various groupings, including the so-called "Devine" system Based on observations by 50 persons of specially designed black and white cards, 2,500 carefully controlled readings were obtained Among the conclusions drawn, it was found that all six-character combinations were superior to seven-digit numbers, that the use of letters increases the difficulty of perception, and that grouping of characters by twos or threes improves perception A gain in visibility seemed to result with six and seven-character plates when the "Devine" system was used No appreciable difference was noted between round and block numerals, but open-type numerals were superior to both Numerals with a width one-half their height were more visible than those with a smaller ratio of width to height The flat-top three was very frequently mistaken for 5, while 8 and 9 also caused considerable confusion

During recent years, the improvement of both automobiles and roads has produced a concurrent rise in average traveling speeds This in turn, causes a need for license plates of higher visibility, yet sufficiently restricted in size so that they will not be unduly expensive, nor improperly illuminated by the tail light at night Regardless of its size, the plate should be as efficient as possible The very large number of cars registered by some states has created legends apparently difficult to perceive and retain, and in many cases, has resulted in crowding of the plate In some cases, various letter-figure combinations are being employed in an attempt to remedy this condition—with varying degrees of success

REQUIREMENTS OF LICENSE PLATES

License plates should identify the vehicle in such a manner as to be easily and accurately read under various traffic conditions and should show that the proper plate is being carried County name, symbols, advertising matter, and other extraneous items tend to reduce legibility and to cause confusion

The identity of the state is usually given by the color combination and the state name Since cars now travel freely among the states, this identification has become of growing importance Professor Wiley in his bulletin "License Plates for Illinois," suggests that the state name should be visible at least half as far as the license number, and in any case, it should be visible for at least 70 feet At present, most plates are very deficient in this respect

It is essential that enforcement officers be able to recognize the proper yearly plate This information is ordinarily given by the color combination and the year in numerals The year should be as visible as the state name, and the color combination should be changed from the previous year Identification of type of vehicle is accomplished usually by key letters in the license number, or by the words "truck," "commercial," "dealer," etc , placed on the plate

The intrinsic legibility of a license

plate will depend upon the combined effect of the following items:

(a) Size and shape of plate.
(b) Height and width of characters.
(c) Style and width of stroke of characters.
(d) Spacing and grouping of characters.
(e) Color combination.
(f) Amount of gloss on the figures and background.
(g) Area ratio of legend to background.
(h) Content and arrangement of the items on the plate.
(i) Number of characters in the legend.
(j) The letters selected for use in combinations.
(k) The number of letters used in the legend.

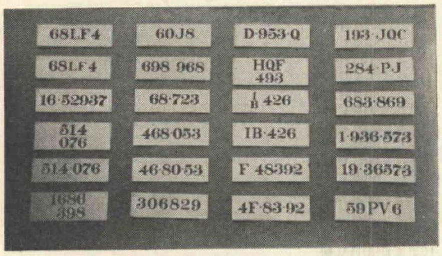


Figure 1

External conditions which will also affect the legibility are:

(a) Distance of the observer from the plate.
(b) Angle at which observer views plate.
(c) Intensity and color of plate illumination.
(d) Angle at which most of the light strikes the plate.
(e) Conditions of vehicle movement (i.e., stationary, slow or high speed, receding or approaching, smooth or rough road, etc.)
(f) Amount of dirt or moisture on the plate.
(g) Atmospheric conditions.
(h) Position of the plate on the vehicle.
(i) Amount of dust which is thrown up by the car, or by passing traffic.

Many of these later items are beyond the control of the plate designer. It has been assumed that the plate which is most legible and most easily perceived when stationary and under varying conditions of daylight will be the most satisfactory under the many varied traffic conditions. This will probably be approximately so except for the matter of color combination. Dr. A. R. Lauer found that certain color combinations which were satisfactory in daylight were indistinct under artificial illumination.[1]

In this paper, investigations are reported with particular reference to:

1. A comparison of the perceptibility of the various systems of numbering used by states whose registrations exceed one million passenger cars.

2. The effect on perception of the different methods of grouping the characters, including the proposed "Devine" system.

3. The effect on perception and retention of introducing one or more letters in the legend.

4. A comparison of the visibility of different styles of numerals.

5. Limited studies of width of stroke, and its effect on the visibility.

PERCEPTION TESTS

Purpose: The object of these tests was to compare the merits of the plain numeral and the various letter-numeral systems grouped in different ways. The effect of using letters the same size, or either larger or smaller than the numerals in the group, was experimented with. So far as possible the many variables with which the experimenter finds himself confronted in this type of investigation were eliminated.

Apparatus: For significant comparisons, it was necessary that the variables of size, style and stroke of the characters, spacers, borders, symbols, and color comminations be eliminated in the tests. The use of actual plates, therefore, was not possible. Cards were made using black characters 2 in. high on a white background. A few of the cards used are illustrated in Figure 1.

The device used for displaying the cards to the observer for a carefully controlled interval of time is illustrated in

[1] Unpublished Bulletin, "A Psychological Analysis of the Legibility of Automobile License Plates."

Figures 2 to 4. A 4- by 6-ft. piece of $\frac{3}{4}$-in. plywood is held in a vertical position by strap iron braces. The display window was cut out, and slotted guides were attached to the rear side of the board to permit the insertion of the cards. The arm "A" (Fig. 2) was cut pivot "E." A bushing in the rocker arm permits the shaft "F" to revolve. This shaft passes through a short curved slot in the board (not visible in the pictures) thus allowing a small vertical movement of the rocker arm. A plywood cam "G" is rigidly attached to the shaft on the

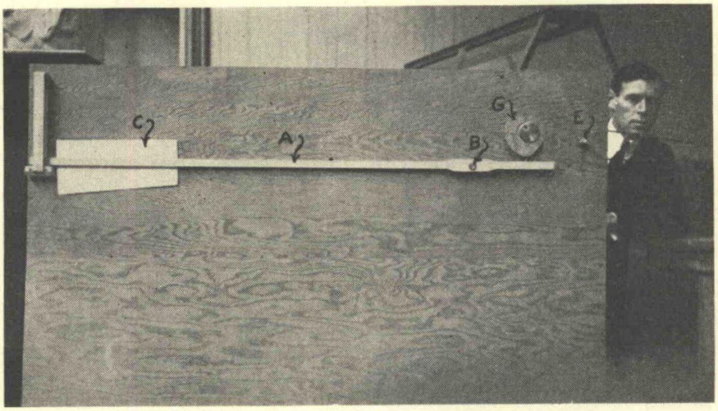


Figure 2

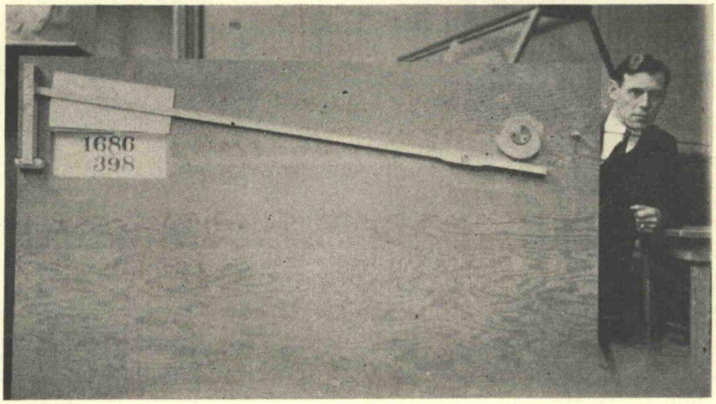


Figure 3

from hard wood, and mounted to the board by means of the pivot "B." The left end of the arm travels in a guide to prevent any tendency to horizontal movement. The screen "C" is attached to the arm, and by this means, the display cards are concealed and exposed as desired. The rocker arm "D" (Fig. 4) is attached to the board by means of the front side of the board, and a 5-inch pulley is similarly attached on the rear side.

In Figure 4 can be seen the apparatus for operating the device. A $\frac{1}{6}$ H. P. 110-volt direct current motor was connected to the 16 to 1 speed reducer "J," by means of a flexible shaft. A 5-in. pulley was mounted on the vertical shaft of the reducer, and belted to the 5-in. pulley "H" at the rocker arm. A rheostat in series with the armature of the motor permitted control of the motor speed from about 200 r. p. m. to 1200 r. p. m.

The spring "K" (Fig. 4) normally holds the rocker arm in its extreme high position. In this position, the cam completely clears the tracer on the right end of the shield arm so that the shaft may revolve continuously without exposing the card. An arrow inscribed on the back of the board together with the shaded portion of the pulley (Fig. 4) indicates to the operator when the small circumference of the cam is directly over the tracer on the shield arm. At this instant, the operator may press down the rocker arm to its low position. The arm is held down for one revolution of the cam, and then released. The card is thus exposed for a carefully controlled interval of time depending upon the speed of the motor.

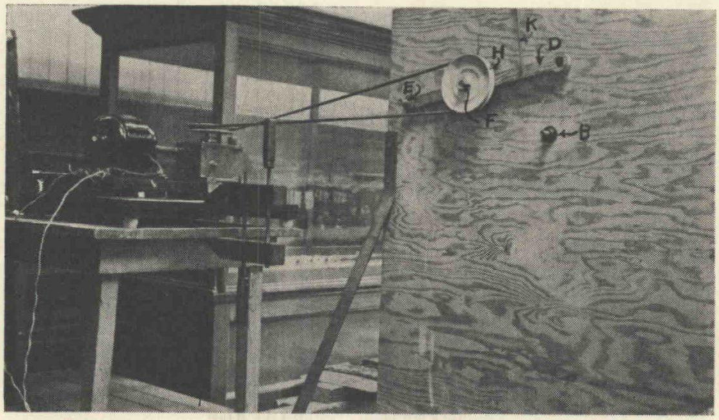


Figure 4

Combinations Tested: Several states have registrations which total over a million passenger cars. Illinois employs the straight numerical system, thus having many plates with seven digits in the legend. New York, California, Michigan and Texas use one letter in a fixed position, thus reducing the maximum number of characters to six. The total number of characters on Pennsylvania and Ohio plates is reduced to five by the use of two letters in combination with numerals.

In a system proposed by Mr. A. W. Devine, Assistant Registrar in Massachusetts, license numbers would be grouped in threes, and the second group placed *below* the first. Thus the number 123,456 would appear $\frac{123}{456}$. Similarly, this could be applied to seven-digit numbers, such a legend appearing as $\frac{1234}{567}$. An application of the "Devine" system has been proposed in which two groups of three characters each would appear on *all* plates. The upper group would always consist of letters, and the lower group of numerals. In this way, certain letters could be used to give information as to the passenger car or truck classification. At least two samples of each of these types were tested.

Arrangement of characters was investigated. Six-digit numbers were grouped in twos, threes, and spaced equally. Seven digits arranged in the form 12-34567 were included. This occurs on Iowa plates since each of the 99 counties is allotted a "county number," which is followed by the car number Seven digits as arranged on Illinois plates were also included

The effect of the relative positions of the letter- and number-groups was tested by placing the letter-group both first and last, and also by inserting the number group between the letters (as in Ohio and Pennsylvania) Some other combinations were included for comparison purposes

Method of Testing· An attempt was made to preclude or at least reduce to a minimum all the possible variables that enter into a psychological investigation of this kind It will be understood, for example, that one group of six digits might be much more difficult to perceive and retain correctly than another group of six, due to the particular digits used in the two groups In testing the effect of systems of grouping, therefore, it would be erroneous to compare the number 698 968 grouped in threes with the number 11·22·33 grouped in twos, or with the number 654321 spaced equally To eliminate this condition, the fifty cards were carefully arranged in two groups, designated "A" and "B" For testing the effect of grouping, one number would appear as 638 492 in the "A" group, and the *same* number would appear differently grouped in "B," viz, 63·84·92 The two groups were perceived by the *same* observers, with an interval of approximately a week between the two tests to obviate the possibility of memory affecting the results Differences in perception could then be attributed mainly to the effect of the grouping

As an observer proceeds through the test, he will gradually improve somewhat in his perception due to practice It is hoped that this factor was minimized to a negligible amount by starting the series of cards at different points for various observers, so that what might be the first card for one observer would come near the middle or end of the series for another Also, about half of the observers were given the "A" group first, and then the "B" group the following week The other half viewed the two series in the reverse order To further eliminate this factor, and also furnish an additional check on the results, a counter number was used for each item tested For example, if the number appeared in the form 382·465 in the "A" series, and the form $\frac{382}{465}$ in the "B" series, then a different number would appear as $\frac{963}{287}$ in the "A," and as 963·287 in the "B" Similar procedure was employed for the various items that were to be investigated

Preliminary tests were made with several different observers to determine a suitable speed at which to operate the mechanism It was decided to operate the motor at 640 r p m, thereby producing a cam speed of one revolution in $1\frac{1}{2}$ min The cam was designed to expose the card during 45 per cent of its revolution The interval of card exposure is thus calculated to be 0 67 sec

The observer was seated at such distance from the board that the cards would be easily visible A test sheet numbered from 1 to 25 was provided The following instructions were read

"Various number and letter groupings as appear on existing license plates and some proposed groupings will be placed in the opening behind the shield The shield will be opened for a short interval of time, and you are to observe and record the legend which appears"

"Regardless of the arrangement on the card, you are to record the legend in a straight line For example (showing a sample of Connecticut plate $\frac{\text{B}}{\text{X}}$288) this should be recorded as BX288, and (showing a sample of the Devine system $\frac{377}{398}$) this is to be recorded as 377398 Any dots or dashes which may appear are to be neglected"

TABLE 1

SUMMARY OF RESULTS OF PERCEPTION TESTS

"A" group					"B" group				
Number	C	N	%W	%C	Number	C	N	%W	%C
36 286	2	1	2	98	68LF4	9	5	10	90
F48392	42	20	40	60	60J8	0	0	0	100
593 652	25	15	30	70	468 053	15	7	14	86
DQ 953	32	16	32	68	193 JQC	16	11	22	78
1 652 937	98	35	70	30	1686 398	108	40	80	20
68LF4	5	5	10	90	D 953 Q	22	9	18	82
306 829	3	3	6	94	698 968	69	23	46	54
LZP 047	43	20	40	60	HQF 493	42	20	40	60
46 80 53	23	11	22	78	284 PJ	2	2	4	96
P 284 J	8	5	10	90	16 52937	72	31	62	38
T D385	5	3	6	94	329E	1	1	2	98
514 076	13	6	12	88	683 869	48	20	40	60
3E29	1	1	2	98	68 723	2	1	2	98
59PV6	10	4	8	92	I B426	9	5	10	90
1686 398	113	39	78	22	1936 573	98	35	70	30
9ZT63	18	10	20	80	9ZT63	19	12	24	76
HQF 493	67	29	58	42	514 076	2	2	4	96
683 869	31	14	28	72	LZP 047	48	25	50	50
J608	2	1	2	98	59 36 52	34	16	32	68
1306 536	23	13	26	74	TD385	6	4	8	92
9E 27 04	11	7	14	86	1306 536	44	19	38	62
19 36573	107	34	68	32	4F 83 92	28	16	32	68
IB 426	2	1	2	98	306829	11	7	14	86
698968	81	32	64	36	59PV6	12	6	12	88
JQC 193	41	23	46	54	E92704	28	15	30	70

KEY

C = Total number of individual characters wrong or omitted
N = Total number of times the legend was perceived incorrectly
%W = Per cent of the time the legend was perceived incorrectly
%C = Per cent of the time the legend was perceived correctly

"I will call 'READY' as a warning each time before the shield is to be opened You will observe the card, and then I will give you sufficient time to record the number Try your best to get the correct legend"

The 25 cards were successively inserted in the opening and the recordings made A week later, the same procedure was followed with the remaining series of cards Fifty individuals were tested in this way, making a total of 2,500 readings Most of the observers were students at the University of Vermont, but some older people from various walks of life were also included

Table 1 gives a summary of the results obtained from the 100 test sheets

Effect of the Number of Characters As will be observed from these data, the particular combination of digits used in a group will greatly affect its correct perception and retention Certain number groupings possess inherent characteristics which render them difficult to perceive correctly Thus, the number "306,829" was correct 90 per cent of the time, while the similar group "698,968" fell as low as 55 per cent The large number of errors in the latter were due mainly to the fact that the observer tended to record the same digits of the two groups in similar sequence, namely, "698,698" However, in comparing the relative perceptibility of 5-, 6-, and 7-digit groups, the use of several groups of each kind will tend to reduce this effect, and seem to justify the following discussion

The percentage of correct readings of the all-numeral groups are given in Table 2

Thus, for the short observational period of a fraction of a second, which would be the case when attempting to obtain the license number of a speeding car, it may be seen that the difficulty of perception increases greatly with the increase in the number of characters The 5-digit groups were read correctly practically all the time while those with 6 digits were perceived correctly only three-fourths of the time When the digits were increased to seven, the correct readings fell off to considerably less than half the total Other things being equal, 6-digit plates are decidedly superior to those with 7, and 5-digit plates in turn are much more easily perceived than those with 6

Assuming an observational period of 0 67 second, little benefit apparently would be gained by an attempt to reduce five-digit plates to four

Effect of Use of Letters Many states have introduced letters into license numbers, either for classification purposes, or to reduce the number of characters

TABLE 2

	Number of groups tested	Per cent read correctly
5-digit	2	98
6-digit	12	74
7-digit	8	39

Some disadvantages may result from this practice Supposing, for example, that one character has been missed in an attempt to catch the number of a speeding car If the plate contains numerals only, the car must be one of ten If 24 letters are also used, the vehicle may be one of 34, thus greatly increasing the difficulty in tracing the car The similarity of some letters to certain other letters and numerals introduces more elements of possible confusion Obviously, certain letters are inherently poor for this reason, and should not be used

License numbers are frequently reported by telephone or radio Hence, the question of audible confusion between letters of similar sound should be considered The selection of proper letters for use will be discussed later

It is claimed that letters on license plates increase the perception time This was tested by comparing groups containing one, two, or three letters with all-numeral groups Again, the variable due to the different characters used can be minimized only by employing several groups of each type for comparison

The average percentages correct for all the groups of each type were computed, and found to be as in Table 3

It appears reasonable to conclude that the introduction of letters increases errors in perception, and that the errors increase in proportion to the number of letters used

To study further the combined effects of the number of characters in the legend, and the use of one or more letters, Figure 5 was constructed From Table 2, the percentages correct for the all-numeral groups were plotted and connected with a full black line From Table 3, it was found that the inclusion of one letter in a 6-character group increased the perception error 5 per cent It was assumed that the introduction of one letter in a 5- or 7-character group would have a similar effect This may not be strictly true, but probably is nearly so Groups including one letter are plotted with a dotted black line Similar procedure was followed with groups containing two and three letters, and plotted with the dash line and the dot-dash line respectively

It should be remembered that these are average values for several numbers grouped in different ways The method of grouping will to some extent affect the perception of any one type However, since approximately the same methods of grouping were employed for the numbers in either the 5-, 6-, or 7-character class, the above comparative results seem justified

From these data, it may be noted that any 6-character group is superior to all 7-character groups Six-character plates containing one letter (used by California, Michigan, and New York) should be perceived correctly 32 per cent more of the time than the 7-digit plates (used by Illinois, and to some extent by Iowa) Five character plates with two letters (Ohio and Penn) show 16 per cent improvement over the 6-character style just mentioned, and are 48 per cent better than the 7-digit groups No advantage in perception would be gained by the introduction of a letter to reduce 5-digit plates to four characters Both types were read correctly 98 per cent of the time The disadvantages involved with the use of a letter seem to outweigh any advantage of possible plate size reduction

TABLE 3

Characters used in the group	Number of groups tested	Read correctly	Error increase due to letters
		%	%
6 numerals—0 letters	12	76	
5 numerals—1 letter	4	71	5
3 numerals—3 letters	6	57	19
5 numerals—0 letters	2	98	
3 numerals—2 letters	14	87	11
3 numerals—1 letter	4	98	

Effect of Method of Grouping With the use of the straight numerical system, the normal method of grouping has been by threes Thus, the 7-digit Illinois plate appears as "1 · 234 · 567 " The system of employing distinguishing county numbers followed by the vehicle number has brought about a different grouping in some states The same number would appear on Iowa plates as 12 34567 The introduction

TABLE 4

Group	Number of times read incorrectly	Per cent read correctly
698 968	23	54
698968	32	36
306 829	3	94
306829	7	86
593 652	15	70
59 36 52	16	68
468 053	7	86
46 80 53	11	78

Average for the legends spaced equally	61%
Average for the same legends grouped in threes	74%
Average for the legends grouped in twos	73%
Average for the same legends grouped in threes	78%

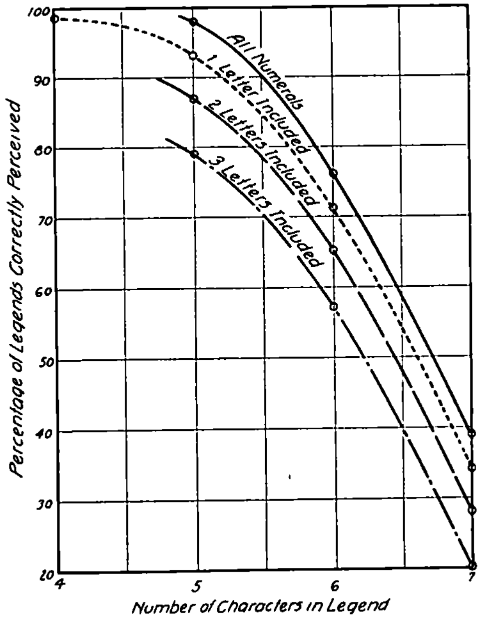


Figure 5 Curves Showing Effect on Correct Perception of Number of Characters in Legend and Number of Letters Included in the Legend

TABLE 5

State	Group	Number of times read incorrectly	Per cent read correctly
Michigan	F 48392	20	60
N Y and California	4F 83 92	16	68
Michigan	E 92704	15	70
N Y and California	9E 27 04	7	86

Average for Michigan type	65%
Average for N Y and California type	77%

of letters has led to other variations California and New York use three groups of twos, thus "5E 46 · 82," while Michigan separates the letter from the numerals, thus "F 17785" (not consistently, as there is no separation on many Michigan plates) On the 2 letter-3 numeral Ohio plates, one will note the following arrangements AB · 345, 345 · AB, A · 345 · B, 3 · AB · 45, and 34 · AB · 5 Pennsylvania and New Jersey do not separate the characters into groups

A comparison was made of the perception due to different ways of grouping In order to eliminate the variable due to the particular characters appearing in the group, one arrangement of certain characters was used in series "A," and a different grouping of the same characters was placed in series "B" The results of 6-digit numbers grouped in twos, threes, and equally spaced are in Table 4

The best grouping for 6-digit numbers appears to be by threes This result would be expected, as this is the normal method of grouping derived from our decimal system However, the difference between grouping by threes and by twos may not be enough to be significant Preference between these two ways probably varies with the individual, some remembering easier by twos, and others by threes Six digits grouped either by twos or threes are quite superior to the same equally spaced

A comparison of the groupings employed by California, New York, and Michigan is shown in Table 5

Some advantage is to be gained by grouping in twos rather than merely separating the letter from the numerals This seems in accord with the previous findings, as the Michigan type is in effect a 5-digit number spaced equally, with an additional character to observe

Table 6 shows a comparison of the 7-digit groupings used on Illinois and Iowa plates

The small difference between the two seems to warrant no definite conclusions Suffice it to state that 7-digit numbers grouped in any way are not readily perceived

Connecticut plates are unique for two reasons First, they are "permanent," and are not renewed annually Secondly, the system of numbering is different from that of any other state, the legend consists of two letters, one placed over the other, followed by three numerals A serious objection to this arrangement is that the letters are made much smaller than the numerals, thus materially reducing visibility

Comparisons were made with groups arranged in this manner, and the same groups arranged in the conventional

TABLE 6

State	Group	Number of times read incorrectly	Per cent read correctly
Illinois	1 652 937	35	30
Iowa	16 52937	31	38
Illinois	1 936 573	35	30
Iowa	19 36573	34	32
Average for Illinois type			30%
Average for Iowa type			35%

TABLE 7

Group	Number of times read incorrectly	Per cent read correctly
IB 426	1	98
${}^{\mathrm{I}}_{\mathrm{B}}426$	5	90
TD385	4	92
${}^{\mathrm{T}}_{\mathrm{D}}385$	3	94
Average for conventional form		95%
Average for Connecticut form		92%

TABLE 8

Group	Number of times read incorrectly	Per cent read correctly
(Penna) 68LF4	5	90
68LF4	5	90
(Penna) 9ZT63	12	76
9ZT63	10	80
59PV6	4	92
59PV6	6	88

form The numbers used were as given in Table 7

The results are insignificant as far as perception is concerned

Pennsylvania uses combinations of letters and numerals A maximum of two letters appear on the plate These are always placed together, but may appear in any position in the legend The letters are made smaller than the numerals Such practice is diametrically opposed to the recommendations of Professor Wiley It is claimed that letters are inherently less legible than numerals, and therefore should be made larger Comparisons of the perception of such groups were made, employing letters larger than, smaller than, and the same size as the numerals These are tabulated in Table 8

No appreciable difference was noted in the perception of the legend due to the relative size of the letters From the standpoint of legibility, they should be made at least as large as the numerals

Devine System of Grouping In 1936, Mr A W Devine, Assistant Motor Vehicle Registrar in Massachusetts, suggested a form of grouping which departs considerably from conventional methods now used Essentially, the license number is arranged in *two* lines of three characters each The year and state name are placed vertically at the ends of the plate It is suggested that letters only be used in the top row, and numerals in the bottom, but there seems to be no reason why the arrangement could not be applied to all numeral legends The fol-

TABLE 9

Group	Number of times read incorrectly	Percent read correctly
514 076	6	88
514 076	2	96
683 869	20	60
683 869	14	72
1686 398	39	22
1686 398	40	20
1306 536	19	62
1306 536	13	74
LZP 047	25	50
LZP 047	20	60
HQF 493	29	42
HQF 493	20	60

		Average % read correctly
6 characters—all numerals	conventional	74
6 characters—all numerals	Devine	84
7 characters—all numerals	conventional	42
7 characters—all numerals	Devine	47
6 characters—including 3 letters	conventional	46
6 characters—including 3 letters	Devine	60
All groups with conventional arrangement		54
All groups with Devine arrangement		64

lowing advantages are claimed for this system

1 The plates would be considerably shorter, but somewhat wider than most of the present ones This should provide for more satisfactory illumination from the tail light

2 All plates would be the same size—an advantage in manufacture, mailing, and mounting on the car

3 This license number is arranged on a "spot" form instead of an extended line It should be more easily taken in by the eye

4 The group of letters is distinctly separated from the group of numerals People would soon learn that letters only appear in the upper row, hence transposition of letters with figures would be reduced

Comparisons between the "Devine" and the conventional grouping were made to determine whether accuracy of perception would be increased by this proposed form The numbers tested, and the results are given in Table 9

There are indications that some advantage in ease of perception is to be gained by the use of the suggested Devine system The difference is not overwhelming, averaging 10 per cent for all the groups It should be noted, however, that this radical departure from the conventional method of displaying numbers might show a decided increase in ease of perception when people became more accustomed to seeing this method of grouping With the other advantages of plate size previously mentioned, the system may have considerable merit, and deserves serious consideration The chief objection to the 3 letter-3 numeral form is its low perception value In this respect, it would be far inferior to any existing plates, save the 7-digit numbers of Illinois and Iowa Furthermore, many three-letter combinations would arise which would form ludicrous or undesirable words, and should be omitted

SUMMARY OF TEST DATA

1 All combinations of 6 characters were superior to 7-digit numbers Five-character legends, in turn, were better than those with 6 There is a greater average difference between 6-character and 7-character groups (34%) than between those with 5 and with 6 (22%)

2 The presence of letters in the group increases the difficulty of perception, this difference increasing with the number of letters appearing

3 There is not much difference between 6-digit plates grouped by threes or by twos, but either is superior to those with the digits spaced equally

4 Six-character plates containing one letter are better when grouped by twos than when the letter only is separated from the rest of the number

5 On 5-character plates containing 2 letters and 3 numerals, it may be slightly preferable to place the letters and numerals in separate groups, although no marked difference was observed

6 An attempt to contrast the letters with the numerals by the use of small letters (Penna and Conn) does not aid in the perception of the number From the standpoint of legibility, the letters should be made at least as large as the numerals

7 Some advantage in perception seems to be gained with 6- and 7-character plates if the "Devine" system of grouping is employed The greatest difference occurs when letters are used in the top row, and numerals in the bottom It is doubtful if any benefits would accrue if used for plates with less than six characters

SELECTION OF LETTERS FOR USE ON LICENSE PLATES

In many states, the use of letters on license plates is a necessary expedient if the legends are to be restricted to a reasonable length The use of key letters also aids materially in the identification of certain classes of vehicles. Their use seems to offer no serious objections, provided the letters to be so employed are selected with care. Confusion arises between letters more frequently than between numerals because there are many more of them and because many bear a resemblance to other letters, or to certain numerals. No extensive investigation on letter confusion seems to have been made, but Dr. Lauer says the letters A, F, H, K, M, R, S, U, W, X, Y, and Z appear to be best.

VISIBILITY TESTS

In connection with a study of Vermont license plates made for the Motor Vehicle Commissioner of that State, limited tests were made of the visibility of several styles of numerals. This was done essentially for the purpose of comparing the numerals used on the present Vermont plates with others of the same height designed for greater legibility, but the results seem to be of sufficient general interest to be included in this paper.

Types of Numerals Compared: Four styles of numerals were tested for visibility: round, block, open, and round as used on the Vermont 1937 plates. To eliminate variables of color, size, spacing, etc., cards were used having black figures on a white background. Figure 6 illustrates some of the combinations used in the test. All numerals were made 3 in. high, the present height of figures on Vermont plates. Numerals are $1\frac{1}{2}$ in. wide, with the exception of the Vermont style which are $1\frac{1}{4}$ in. Two widths of stroke $\frac{3}{8}$ in. and $\frac{1}{4}$ in. were employed. The round and block numerals represent the styles in most common use at the present time. The open style is an attempt to design a style which will emphasize the distinctive features of each numeral with the hope of reducing possible confusion of various digits to a minimum. Styles somewhat resembling this open type have already been used by several states, notably New York, Pennsylvania, and the District of Columbia, but in most cases, the figures would seem to be too narrow for their relative height.

Method of Testing: The figures were arranged in groups of five or six to simulate the arrangement on the majority of license plates, and placed on a back-

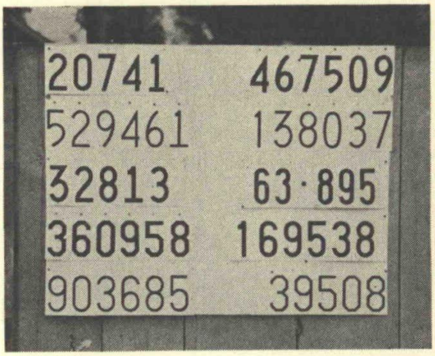


Figure 6. Showing Styles of Numerals Tested

In the order illustrated, they are as follows:

1. **Round style, $\frac{3}{8}$-in. stroke**
2. **Open style, $\frac{1}{4}$-in. stroke**
3. **Open style, $\frac{3}{8}$-in. stroke**
4. **Block style, $\frac{3}{8}$-in. stroke**
5. **Round style, $\frac{1}{4}$-in. stroke**
6. **Open style, $\frac{3}{8}$-in. stroke**
7. **Open style, $\frac{1}{4}$-in. stroke**
8. **Vermont 1937, $\frac{3}{8}$-in. stroke**
9. **Round style, $\frac{3}{8}$-in. stroke**
10. **Block style, $\frac{1}{4}$-in. stroke**

ground as shown in Figure 6. Stakes were placed at 10-ft. intervals from the display of figures up to a maximum distance of 200 ft., and marked with the proper distance. The observer was first placed at the 200-ft. point, and asked to read the groups of digits as they appeared to him at that distance. Those that were very indistinct were omitted. His readings were recorded. He then moved up to the 190-ft. point and repeated the process. At each point, however, the readings were made in a different order to eliminate as far as possible variations due to the effect of memory or eye-fatigue. This process was continued until any one group was read correctly at two consecutive stations. Said group was then considered to be visible to the observer at the more distant of the two points, and was not considered further. The observer thus moved forward in 10-ft increments until all groups had been read correctly.

in place of the correct digit which appears at the head of the column. In the open type groups, both a flat top three and the rounded three were included. In one of the open type and one of the round type groups, the digit "1" both with and without a base was included. Block and Vermont type numerals not listed in this table were not included in the test.

Style of Numerals. In order to determine the effect of the style of numerals

TABLE 10

SHOWING THE MAXIMUM DISTANCES AT WHICH THE VARIOUS NUMERAL GROUPS WERE READ BY EACH OBSERVER

Observer number	Open style $\frac{3}{8}$-in stroke	Open style $\frac{1}{4}$-in stroke	Round style $\frac{3}{8}$-in stroke	Block style $\frac{3}{8}$-in stroke	Block style $\frac{1}{4}$-in stroke	Round style $\frac{1}{4}$-in stroke	Vermont style $\frac{3}{8}$-in stroke
	ft	*ft*	*ft*	*ft*	*ft*	*ft*	*ft*
1	150	150	130	120	130	120	120
2	190	190	160	170	160	140	160
3	100	100	100	80	90	90	80
4	160	140	150	130	130	130	110
5	160	150	120	130	130	120	120
6	180	170	160	160	150	160	160
7	160	150	150	150	160	140	150
8	130	130	130	120	120	110	120
9	160	150	160	150	150	150	140
10	160	150	150	150	140	150	130
11	160	160	140	140	150	140	140
12	130	140	120	120	120	110	120
13	150	150	150	150	140	140	130
14	170	160	160	160	150	150	130
Average	154	150	141	138	137	132	129

These tests were made outdoors under varying conditions of weather, to correspond somewhat with actual traffic conditions in daylight. No tests were made with artificial illumination. Fourteen observers were tested in this way.

Tabulation of Results. Table 10 was compiled from the test sheets. It shows the maximum distance at which a group of each style and width of stroke was visible to each observer. From these readings, the average visibility distance for each type of numeral was calculated.

The confusion of digits is shown in Table 11. The numbers appearing in the columns are those given by the observers

on visibility, variations of height, width, and stroke must be eliminated. These factors were the same for the open, round, and block types with $\frac{3}{8}$-in stroke, and also for the same three styles with $\frac{1}{4}$-in stroke. From Table 10, the average visibility distance of these groups is as given in Table 12.

No appreciable difference in visibility is found between the round and block styles. This result might be expected, as the angular corners of the block figures become rounded to the eye when viewed at a distance, and the numerals then closely resemble the round type in appearance.

The open style seems to show a decided advantage in visibility for the numerals with both the $\frac{3}{8}$- and $\frac{1}{4}$-in stroke Other factors being the same, the open type of numeral is more visible than either of the much-used round or block types

The digit one with a base was found to be visible at an average distance of 194 ft as compared with 174 ft for the one without a base The former should be used The rounded three was visible at a greater distance than the flat top

TABLE 11

Comparison of Confusion Between the Digits of the Various Styles of Numerals

Numerals heading the columns are the ones that should have been given Below each of these are listed the digits that were actually given by the observers

Open type $\frac{3}{8}$-in stroke

1	2	3	3	4	5	6	7	8	9	0
	1	1	5	5	6	1		3	7	6
	1	5			6	1		7	8	
	1	5			8	1		7	8	
	3	5			0	0		9	8	
	3	5						9	0	
	3	5								
	7	5								
	7	5								
	8	5								
		5								
		5								
		5								
		5								
		5								
		5								
		5								
		5								
		5								
		5								
		5								
		5								
		6								

Open type $\frac{1}{4}$-in stroke

1	1	2	3[a]	3[b]	4	5	6	7	8	9	0
			4	1	6		4		0	6	
			5	0			4			6	
			5				4			6	
			5				8			6	
			5							8	
			5								
			5								
			5								
			5								

Round type $\frac{3}{8}$-in stroke

1	1	2	3	4	5	6	7	8	9	0
		3	1	0	3		6	3	2	3
		5	1	0			6	9	3	7
		8	1					9	3	7
		8	1					9	8	7
		8	2					9	8	8
			5					9		8
			7					9		9
			7					9		9
			7					9		
			7							
			7							
			7							
			7							

Round type $\frac{1}{4}$-in stroke

3	5	6	8	9	0
6		8	4	1	
8		8	5	2	
			5	3	
			5	3	
			6		
			6		
			6		
			6		
			9		

[a] Flat top
[b] Round

TABLE 11—*Continued*

Block type ⅜-in stroke						Block type ¼-in stroke					Vermont type ⅜-in stroke				
3	5	6	8	9	0	3	5	8	9	0	3	5	6	8	9
8		4	1	5	8		8	9	3		1	1	1	4	1
		4	6	8			9	9	3		1	3	4	4	1
		8	6	8			0	9	5		1		4	4	1
			9	8				9	5		1			5	1
			9	0				9	5		1			7	1
			9	0				9	5		1			9	1
			9					9	5		1			0	1
			0					0	5		1			0	2
			0						5		1			0	3
			0						5		1				4
			0						5		2				5
			0						7		7				5
			0						8		7				7
			0						8		7				7
									8		7				7
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											7				0
											7				0
											9				0
											9				0
											0				0
															0
															0
															0

TABLE 12

⅜-in stroke		¼-in stroke	
Type	Average visibility	Type	Average visibility
	ft		*ft*
Open	154	Open	150
Round	141	Round	132
Block	138	Block	137

three, although the low legibility of the latter was due chiefly to its confusion with "5"

Width of Stroke In all cases, the numerals with ⅜-in stroke were superior to those of the same style with ¼-in stroke The difference was not phenomenal, however, being only 4 ft for the open type, 9 ft for the round, and 1 ft for the block numerals Since only two widths of stroke were employed in this experiment, nothing can be concluded as to the best width of stroke for a given set of numerals The optimum stroke doubtless would vary with the style, height, and width of the numerals, the ratio of height to width, and possibly other factors It is a question which could well be investigated further

Ratio of Width to Height: Only two

widths of numerals were employed The round style with $\frac{3}{8}$-in stroke are $1\frac{1}{2}$-in wide, and the Vermont type are similar in all respects except their width, which is $1\frac{1}{4}$ in The round style were visible at an average distance of 141 ft while the Vermont numerals dropped to 129 ft Therefore, 3 by $1\frac{1}{2}$-in numerals are quite superior to 3 by $1\frac{1}{4}$-in numerals

From a practical standpoint, the optimum ratio of width to height for license plate numbers involves other items such as the available plate size, and the spacing and grouping of the numerals Quite extensive tests would be required to investigate this matter For the space available on the Vermont plate, whose size is prescribed by law, it was found that numerals $1\frac{1}{2}$-in wide were definitely superior to the $1\frac{1}{4}$-in ones used at present

Confusion of Digits In all the groups tested, little confusion was found to result with the digits 2, 4, 5, 6, 7, and 0 The digit "2" was confused to some extent, being given several times as 1, 3, 5, 7, and 8 The block "2" was not confused at all There seems to be no explanation why this should be more distinct than either the open or the round "2 "

The flat top three is very bad It appeared only twice in the display of figures, and yet it was given as "5" a total of 28 times It is certain that this type of three should never be used The more rounded three was infrequently confused except in the case of the somewhat narrow Vermont numerals In this case, it was given frequently as "1" or "7," probably due to the compressed nature of these figures These same digits were given to some extent for the round style three with $\frac{3}{8}$-in stroke, a fact which is not easily explained

The digit "8" was confused considerably, being given most often as "9 " This was particularly true for the block and round types where the "9" is differentiated from the "8" only by the small opening near the bottom of the "9 " The open style "9" should tend to reduce this confusion

The digit "9" was confused more often than any other It was given most frequently as "5" or as "8 " For the narrow Vermont numerals, it was also given frequently as 1, 7, and 0 It is a very curious fact that "9" is the digit most frequently confused, while "6" is one of those least confused Dr Lauer obtained the same results in his tachistoscopic studies with numerals, and explains this apparent paradox in this way "It may seem strange that nine is the most confused and six, which is an inverted nine, the least confused This very interesting result may be explained by the fact that most cues in reading are secured from the upper half of the letters or characters One can observe this for himself by placing a paper over the upper half of the letters and attempting to read By reversing it so the lower half is covered, no disturbance is noted Modification of the upper portions of the confused digits may make them more legible "

Since all numerals were spaced one half inch apart, no comparisons of legibility due to spacing can be made In this regard, Dr Lauer says, "The correlation with legibility was + 2186 This would be interpreted to mean that greater spacing between the numbers would greatly increase their legibility Based upon measurements of sizes of plates now being used, these letters should not center closer than 2 in It would be better to increase this distance if possible "

It was observed that digits in the end positions of the groups were more legible than when placed elsewhere This fact tends to confirm the argument that wide spacing promotes greater visibility Undoubtedly, the widest spacing of numerals consistent with a reasonable size of plate will be the best

Summary The open type numerals are more visible than either the round or block types. There is no appreciable difference in visibility between the latter two styles.

For numerals 3 in. high and $1\frac{1}{2}$ in. wide, a $\frac{3}{8}$-in. stroke is preferable to a $\frac{1}{4}$-in. stroke, although the difference is not momentous. Further study should be made on the question of best width of stroke.

Numerals whose width is half their height are more visible than those with a

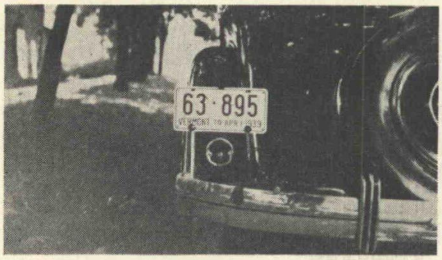

Figure 7

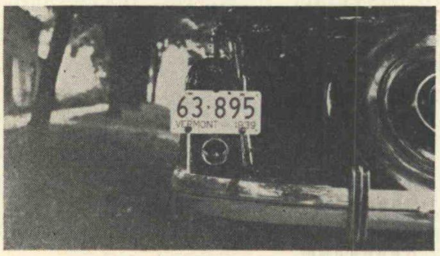

Figure 8

smaller ratio of width to height. For good visibility, license plate numerals should be made wider in respect to height than is generally the practise at present.

The digits 1, 4, 5, 6, 7, and 0 are not seriously confused. The flat top three is very frequently confused with "5" and should never be used. The digit "8" is often confused, especially with "9." Nine is the digit most frequently confused, and is often given as "5" or "8." Since it is hardly feasible to omit any digit even though it may be confused frequently, numerals should be designed to emphasize their distinguishing characteristics. The open type 5, 6, and 9 should be of some aid in this direction. The digit "1" should have a base, but the top should be small lest this digit be confused with "7."

License Plate Design: Illustrations of a few sample license plates submitted to the Motor Vehicle Commissioner of Vermont are here included to show what may be done in the way of plate design to increase visibility. Figure 7 is a replica of the plate used in Vermont for 1937. Its chief objections are: numerals too narrow in proportion to their height, year and state name too crowded and compressed, border unnecessary and cuts down the effective size of the plate, characters along the bottom of the plate partially obscured by the fastenings. The numerals were visible at an average distance of only 129 ft.

The numerals used on the plate shown in Figure 8 are of the open type, the most legible of the four styles tested. Numerals are spaced $\frac{1}{2}$ in. apart, with a $1\frac{3}{4}$ in. space between the groups. The border has been omitted, which increases the effective size of the plate. The letters used for the state name and year are much superior to those shown in Figure 7, but they are still somewhat obscured by the fastenings. The numerals are very well shaped and spaced for high visibility. This type of figures was visible at an average distance of 154 ft.

Figure 9 illustrates the arrangement of the plate which has been adopted in Vermont for 1938. The border has been omitted, however, in order to increase the effective size of the plate. The visibility of the state name and year is thus appreciably increased. This arrangement of these items also possesses the desirable feature that no part of either is partially obscured by the plate fastenings. From all appearances, these plates are far superior to those used last year.

The remaining five digits of the open style are shown on the plate in Figure 10. These may also be used for the round type with the exception of "0" which should be designed with semicircular top and bottom and straight sides. A base has been retained on the "1" as this was found to be more visible than the digit "1" with base omitted. By omitting the border and abbreviating the state name and year, it was possible to increase the size of the characters used for the name and year. Their legibility is considerably increased in this way.

All plates are 6 by $12\frac{1}{2}$ in., the size prescribed for Vermont. All numerals are 3 in. high.

Figure 9

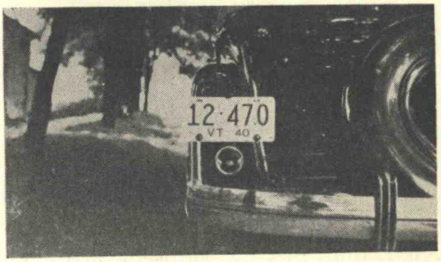

Figure 10

DISCUSSION ON AUTOMOBILE LICENSE PLATES

Mr. Jos. Barnett, *U. S. Bureau of Public Roads:* The visibility of license plates is associated with tail lights and their location. The present location of tail lights is about two feet above the ground, which appears to be very undesirable. Why it was put there possibly is a throwback to the lantern hung from the buckboard when horse-drawn vehicles predominated, but more likely is a throwback to the era of open body types when the top of a vehicle put back against the rear made it necessary to place tail light and license plate low enough to be visible.

At the present time only about one-third of one per cent of vehicles are constructed with open bodies so that the open body type as a factor may be disregarded. Tail lights and license plates on closed body types ought to be placed near the top of the vehicle. In their present location they are in the way of the fenders which appear to require replacing or repairing with increasing frequency; they are in the way of the gas tank; they are in the way of the trunk rack; they are in the way of the spare tire when at the rear; and they frequently are spattered with mud, reducing the efficiency of the tail light and visibility of the license plate.

There also is the important matter of safety. Many night accidents are recorded in which the driver or a passenger obscured the tail light by standing at the rear of the vehicle either to change a tire or to get at the trunk rack, at which time they were ploughed into by following vehicles. Accidents of this character generally result in serious injury or fatality. One of the danger spots on a highway is the crest of a hill over which the sight distance generally is limited. A tail light at the top of a vehicle first is seen at a considerably greater distance than one near the pavement. This safety feature is recognized for trucks and busses on which some kind of light nearly always is located at or near the top. There is just as much justification for this practice for passenger vehicles.

Of course these observations do not detract from the value of Professor Aldrich's excellent work because, regardless of where it is put, the desirability of a license plate which is highly visible cannot be questioned. It does appear, however, that the automobile industry ought to take cognizance of the fact that license plates and tail lights are much more visible at the tops of vehicles than where they now are located

Mr C M Johnston, *U S Bureau of Public Roads*· I should like to make one observation on Mr Barnett's remarks and that is, sometimes the tail light goes out Hence, if you put a tag at the top of the car, the driver of a vehicle approaching from behind might not see the license plate if he wanted to. If you have the plate a little lower, say chest height, then the lights of the approaching vehicle should at least shine on it and thus illuminate it for identification purposes In other words, it is my opinion, that the maximum height of the license plate above the ground should be governed by the maximum elevation of the headlight beam permitted by law and the legible distance of the tag

It might also be observed that an extinguished tail light makes a very good danger signal when reflecting rays of an oncoming headlight This is another reason for not elevating the tail light too high

Dr A R Lauer, Iowa State College Professor Aldrich and I corresponded on this point before Two years ago we succeeded in measuring the reflection factor of different colors on the standard license plates used about 1929-30 A number of factors were studied relating to legibility Professor Aldrich has taken up certain of these more in detail One or two observations may be made relating to his paper While the $\frac{3}{8}$-in may be better than the $\frac{1}{4}$-in stroke we found that if the stroke gets much larger than $\frac{3}{8}$-in , there is a tendency for the number or letter to blur This lowered the efficiency a great deal

Another observation is that his model plates showed the state identifications at the bottom We found this a decided advantage over vertical state identifications In such cases the small letters tended to run together and resemble a letter I or number 1 This point is related to another regarding technique In the tachistoscope shown the shutter moves from the bottom of the number to the top then drops This gives a longer exposure time for the lower part of the number As a matter of fact the upper part of the letter or numeral is used most in visual perception of printed characters Try laying a white card along the bottom line of letters You will note that it does not affect your ease of reading the line Now try covering the upper part of the line and notice how much it interferes with reading Thus, the method of exposure might affect the results considerably, altho this may not be a factor in the present study

It suffices to say that anything at the top of the license plate is likely to reduce the efficiency in visual perception of the number on the plate Smaller identifications should be reduced to a minimum but if put on at all should be at the bottom

Regarding the matter of tail lights placed over the numbers, we found in our studies that the amount of impinging light did not make a great deal of difference, within reasonable limits, when number plates of sufficient contrast between numbers and background were used However, when black numbers were used on a dark green background the illumination had to be quite high for reasonable efficiency of the plates In the latter plate the reflection factor of the numbers was about 4 per cent and that of the background only about 20 per cent The difference was around 15 per cent The best plates for low illumination were those with wide differences between background and number It seems to me that the color combinations of the plates are of even more importance than the shape of letters or numbers, and slight differences in illumination which it is possible to throw upon them.

DISPERSION OF HIGHWAY TRAFFIC BY TIME PERIODS

BY W A SHELTON

Senior Transport Economist, U S Bureau of Public Roads

SYNOPSIS

It has been assumed by many that if sample records of highway traffic be confined to certain hours of the day, days of the week, and months of the year, the dispersion will be less than if the observations are taken at random through the year For Holland Tunnel and George Washington Bridge calculations show that from 8 to 12 in the morning and from 3 to 7 in the afternoon are the best hours of the day For the months, July, June, August and March are the most nearly constant in hourly variations, while April and May and September through November are more highly variable Of the days of the week Wednesday and Saturday are best and Monday and Tuesday most variable

Attention has been given to methods of sampling highway traffic volume and to characteristics of the traffic populations of Holland Tunnel and George Washington Bridge in previous papers[1] In the last of these the analysis of the constants was confined to those measures of the total populations of the routes here named In this article attention is confined to measures of the sub-populations of the same parent populations The sub-populations considered are (1) each hour of the day for the 365 days of the year, (2) all hours of each day of the week (1,248 hours) for the year, and (3) all hours of each month of the year

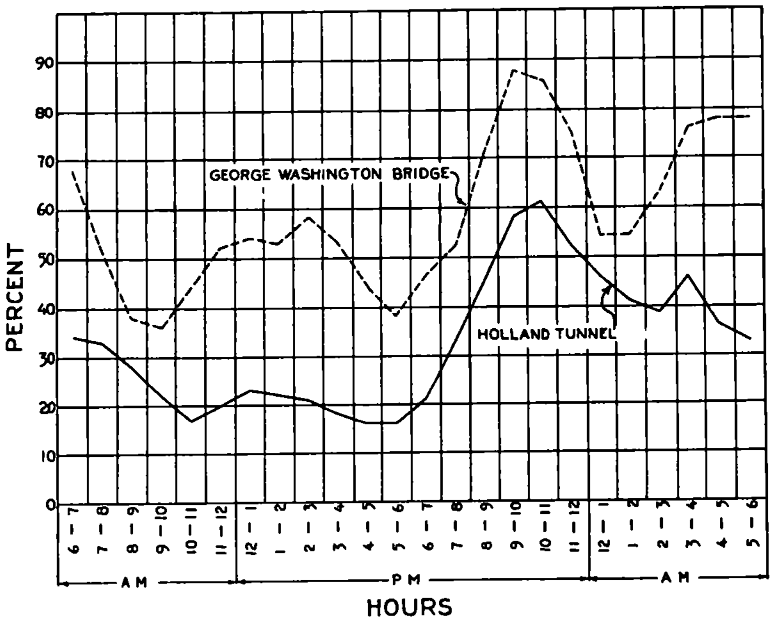

Figure 1 Dispersion of Number of Vehicles by Hours for the Calendar Year 1933 in Chronological Order

The object of computing the measures of these 24 sub-populations of the hours of the day, the seven sub-populations

[1] *Proceedings,* Highway Research Board, Vol 14, p 398 (1934), and Vol 16, p 239 (1936).

of the days of the week, and the 12 of the months was to separate the factors of total dispersion of volume of traffic by hours for the 43 separate sub-populations of each parent population If the dispersion, or scatter, in volume of traffic for each sub-population can be separated, those of smallest dispersion can be ascertained and used as sources of samples with the result of greater precision of estimate than could be obtained from the total population at random

HOURS OF THE DAY

In Figure 1 is shown the dispersion of the number of vehicles by hours for the calendar year 1933 in chronological order for Holland Tunnel and George Washington Bridge The curves represent the coefficients of variation, or the average hourly dispersion for each hour of the day measured in percentage of the mean For Holland Tunnel the smallest dispersion is for the hours from 9 in the morning to 7 at night, a total of 10 hours, with a coefficient not varying far from 20 percent of the mean For George Washington Bridge the corresponding group extends from 8 in the morning to 7 at night, a total of 11 hours, but the dispersion ranges from 36 to 58 percent compared with 16 to 28 percent for Holland Tunnel, the scatter being very

TABLE 1

DISPERSION OF NUMBER OF VEHICLES BY HOURS OF THE DAY FOR THE CALENDAR YEAR 1933 IN CHRONOLOGICAL ORDER

Hour, a m	Holland Tunnel		George Washington Bridge	
	Standard deviation[1]	Coefficient of variation[2]	Standard deviation[1]	Coefficient of variation[2]
	(vehicles)	*(percent)*	*(vehicles)*	*(percent)*
6- 7	179	33 78	168	68 28
7- 8	294	33 34	258	52 10
8- 9	405	27 99	293	38 01
9-10	368	21 62	297	36 41
10-11	308	17 27	380	44 02
11-12	341	19 51	446	52 09
12- 1	379	23 19	451	54 26
1- 2	358	21 79	459	52 78
2- 3	354	20 62	529	58 22
3- 4	325	17 65	525	52 93
4- 5	306	16 29	469	43 52
5- 6	336	15 98	461	38 30
6- 7	399	21 32	530	46 22
7- 8	501	32 92	490	51 77
8- 9	581	45 48	584	70 52
9-10	645	58 22	650	87 65
10-11	659	60 64	590	86 43
11-12	609	51 74	479	74 69
12- 1	392	45 58	253	54 14
1- 2	232	41 05	155	53 97
2- 3	168	39 48	113	63 43
3- 4	158	45 56	97	75 74
4- 5	101	36 03	79	78 18
5- 6	109	32 98	104	78 34

[1] Standard deviation of the 365 hours of each hour of the day for the year

[2] The coefficient of variation is the standard deviation divided by the mean of the 365 hours for each hour of the day

great from 11 in the morning to 3 in the afternoon. For both routes the greatest dispersion is from 9 in the evening to 12 midnight. From 8 to 9 in the evening it is almost as great, and for George Washington Bridge it is also large from 3 to 6 in the morning. The dispersion for the worst hours extends from 45 to 88 percent of the mean, and yet from 8 to 10 in the evening has usually been included in the day period for estimation of traffic volume.

The small dispersion of hourly volume of traffic from 9 in the morning to 7 at night makes it feasible to sample from them with materially smaller error than if samples are drawn from the more widely dispersed hours. For Holland Tunnel the best period of 8 hours is from 10 to 12 in the morning (with 12 to 1 for luncheon) and 1 to 7 in the afternoon (Figure 1 and Table 2). For George Washington Bridge the best period of 8 hours is from 8 to 12 in the morning and from 4 to 8 in the evening, but it might be changed to 3 to 7 with only slightly more error.

Since George Washington Bridge receives part of the overflow from Holland Tunnel, the traffic of the former is more dispersed and that of the latter less dispersed because of the overflow during congested periods of Holland Tunnel. The average dispersion is therefore shown for the hours of the day for the two routes in Table 7. The best average period of 8 hours is from 8 to 12 in the morning and from 3 to 7 in the afternoon. A 7-hour period can be had by omitting the last hour, 6-7, or 11-12 in the morn-

TABLE 2

DISPERSION OF NUMBER OF VEHICLES BY HOURS OF THE DAY FOR THE CALENDAR YEAR 1933 IN ASCENDING ORDER OF THE COEFFICIENT OF VARIATION

Hour	Holland Tunnel		Hour	George Washington Bridge	
	Standard deviation	Coefficient of variation		Standard deviation	Coefficient of variation
	(vehicles)	*(percent)*		*(vehicles)*	*(percent)*
5- 6 p.m.	336	15.98	9-10 a.m.	297	36.41
4- 5 p.m.	306	16.29	8- 9 a.m.	293	38.01
10-11 a.m.	308	17.27	5- 6 p.m.	461	38.30
3- 4 p.m.	325	17.65	4- 5 p.m.	469	43.52
11-12 a.m.	341	19.51	10-11 a.m.	380	44.02
2- 3 p.m.	354	20.62	6- 7 p.m.	530	46.22
6- 7 p.m.	399	21.32	7- 8 p.m.	490	51.77
9-10 a.m.	368	21.62	11-12 a.m.	446	52.09
1- 2 p.m.	358	21.79	7- 8 a.m.	258	52.10
12- 1 p.m.	379	23.19	1- 2 p.m.	459	52.78
8- 9 a.m.	405	27.99	3- 4 p.m.	525	52.93
7- 8 p.m.	501	32.92	1- 2 a.m.	155	53.97
5- 6 a.m.	109	32.98	12- 1 a.m.	253	54.14
7- 8 a.m.	294	33.34	12- 1 p.m.	451	54.26
6- 7 a.m.	179	33.78	2- 3 p.m.	529	58.22
4- 5 a.m.	101	36.03	2- 3 a.m.	113	63.43
2- 3 a.m.	168	39.48	6- 7 a.m.	168	68.28
1- 2 a.m.	232	41.05	8- 9 p.m.	584	70.52
8- 9 p.m.	581	45.48	11-12 p.m.	479	74.69
3- 4 a.m.	158	45.56	3- 4 a.m.	97	75.74
12- 1 a.m.	392	45.58	4- 5 a.m.	79	78.18
11-12 p.m.	609	51.74	5- 6 a.m.	104	78.34
9-10 p.m.	645	58.22	10-11 p.m.	590	86.43
10-11 p.m.	659	60.64	9-10 p.m.	650	87.65

ing The variation of the coefficient of variation from 27 14 to 73 54 percent of the mean for the hours of the day indicates that there would be a large saving in error of estimate by taking samples from the hours in the lower percentage ranges

Saturday (52 23) and the largest for those of Tuesday (56 43) For George Washington Bridge the smallest is for Wednesday (61 14) and the greatest for Monday (70 65) The average for the two routes is smallest for Thursday (57 92) and largest for Tuesday (63 34)

TABLE 3

DISPERSION OF NUMBER OF VEHICLES BY HOURS FOR EACH DAY OF THE WEEK FOR THE CALENDAR YEAR 1933 IN CHRONOLOGICAL ORDER

Day of week	Holland Tunnel		George Washington Bridge	
	Standard deviation[1]	Coefficient of variation[2]	Standard deviation[1]	Coefficient of variation[2]
	(vehicles)	*(percent)*	*(vehicles)*	*(percent)*
Sunday	851	54 30	807	67 97
Monday	644	52 66	433	70 65
Tuesday	616	56 43	368	70 26
Wednesday	620	54 83	325	61 14
Thursday	602	53 32	333	62 52
Friday	674	55 30	383	65 89
Saturday	685	52 23	502	66 91

[1] Standard deviation of the hours of each day of the week for the year

[2] The coefficient of variation is the standard deviation divided by the mean of the sub-population of hours for each day of the week for the year

TABLE 4

DISPERSION OF NUMBER OF VEHICLES BY HOURS FOR EACH DAY OF THE WEEK FOR THE CALENDAR YEAR 1933 IN ASCENDING ORDER OF THE COEFFICIENT OF VARIATION

Day of week	Holland Tunnel		Day of week	George Washington Bridge	
	Standard deviation	Coefficient of variation		Standard deviation	Coefficient of variation
	(vehicles)	*(percent)*		*(vehicles)*	*(percent)*
Saturday	685	52 23	Wednesday	325	61 14
Monday	644	52 66	Thursday	333	62 52
Thursday	602	53 32	Friday	383	65 89
Sunday	851	54 30	Saturday	502	66 91
Wednesday	620	54 83	Sunday	807	67 97
Friday	674	55 30	Tuesday	368	70 26
Tuesday	616	56 43	Monday	433	70 65

DAYS OF THE WEEK

Although some of the hours of the day have very small coefficients of dispersion and others very large ones, all days of the week have relatively large coefficients of hourly dispersion but with no very great range of difference among the 7 days For Holland Tunnel the smallest dispersion is for the hours of Saturday and Sunday stands in the middle of the array For George Washington Bridge Saturday is fourth from the smallest and Sunday fifth This test shows that neither Saturday nor Sunday stands at the top in dispersion for either of the two routes (Tables 3 and 4 and Figure 2)

The common custom of eliminating Saturday and Sunday as the most widely dispersed days by hours is therefore not supported by the coefficients of variation Based on the two routes Tuesday and Monday show the greatest average dispersion of hourly traffic, while Saturday stands third from the smallest and Sunday third from the largest in dispersion (Table 7) The difference in dispersion, moreover, is not great among any of the days. The results indicate that a very slight advantage would be gained by sampling from Thursday, Wednesday, and Saturday or by omitting at least Tuesday and Monday. The variation in dispersion is so small, however, that all days may well be included with little increase in error of estimate

MONTHS OF THE YEAR

The dispersion of hours by the months of the year is rather closely similar for

TABLE 5

Dispersion of Number of Vehicles by Hours for Each Month of the Calendar Year 1933 in Chronological Order

Month	Holland Tunnel		George Washington Bridge	
	Standard deviation[1]	Coefficient of variation[2]	Standard deviation[1]	Coefficient of variation[2]
	(vehicles)	*(percent)*	*(vehicles)*	*(percent)*
January	626	56 11	364	72 04
February	585	56 94	323	71 37
March	586	55 23	309	67 28
April	737	56 05	483	75 96
May	700	53 46	523	73 76
June	683	50 52	546	69 99
July	674	48 44	614	65 78
August	690	51 11	615	68 35
September	752	54 77	623	74 33
October	745	57 33	571	78 11
November	698	57 79	461	74 29
December	653	61 15	396	75 25

[1] Standard deviation of the hours of each month of the year

[2] The coefficient of variation is the standard deviation divided by the mean of the sub-population of hours for each month of the year

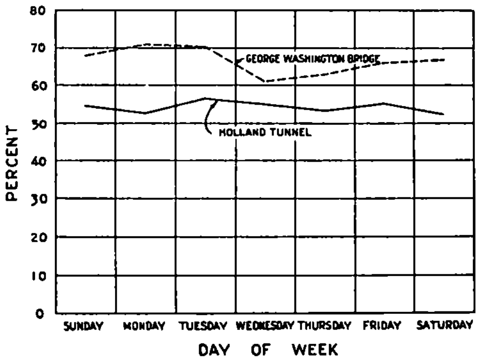

Figure 2 Dispersion of Number of Vehicles by Hours for Each Day of the Week for the Calendar Year 1933 in Chronological Order.

most of the months, and the scatter of the coefficients among the months is not wide (Tables 5-7 and Figure 3). This indicates that no great increase in precision can be made by choosing among the months in timing traffic surveys Because the spring and fall months lie between the extreme seasons of the winter and the summer in volume of traffic, it has been assumed by many that better brief surveys could be made by taking the counts in the spring and fall months, the means of which are near the annual monthly mean It is not the average size of the hour within the

TABLE 6

DISPERSION OF THE NUMBER OF VEHICLES BY HOURS FOR EACH MONTH OF THE CALENDAR YEAR 1933 IN ORDER OF ASCENDING COEFFICIENT OF VARIATION

Month	Holland Tunnel		Month	George Washington Bridge	
	Standard deviation	Coefficient of variation		Standard deviation	Coefficient of variation
	(vehicles)	*(percent)*		*(vehicles)*	*(percent)*
July	674	48 44	July	614	65 78
June	683	50 52	March	309	67 20
August	690	51 11	August	615	68 35
May	700	53 46	June	546	69 99
September	752	54 77	February	323	71 37
March	586	55 23	January	364	72 04
April	737	56 05	May	523	73 76
January	626	56 11	November	461	74 29
February	585	56 94	September	623	74 33
October	745	57 33	December	396	75 25
November	698	57 79	April	483	75 96
December	653	61 15	October	571	78 11

TABLE 7

AVERAGE COEFFICIENTS OF VARIATION OF HOURS FOR EACH SUB-POPULATION OF HOLLAND TUNNEL AND GEORGE WASHINGTON BRIDGE GROUPED BY THE THREE TIME FACTORS OF VARIATION, NAMELY (1) HOURS OF DAY, (2) DAYS OF WEEK, AND (3) MONTHS OF YEAR IN ASCENDING ORDER OF COEFFICIENT

Hour of day	Average coefficient	Day of week	Average coefficient	Month of year	Average coefficient
	(percent)		*(percent)*		*(percent)*
5– 6 p m	27 14	Thursday	57 92	July	57 11
9–10 a m	29 02	Wednesday	57 98	August	59 73
4– 5 p m	29 90	Saturday	59 57	June	60 26
10–11 a m	30 64	Friday	60 60	March	61 26
8– 9 a m	33 00	Sunday	61 14	May	63 61
6– 7 p m	33 77	Monday	61 66	January	64 08
3– 4 p m	35 29	Tuesday	63 34	February	64 16
11–12 a m	35 80			September	64 55
1– 2 p m	37 28			April	66 00
12– 1 p m	38 72			November	66 04
2– 3 p m	39 42			October	67 72
7– 8 p m	42 34			December	68 20
7– 8 a m	42 72				
1– 2 a m	47 51				
12– 1 a m	49 86				
6– 7 a m	51 03				
2– 3 a m	51 46				
5– 6 a m	55 66				
4– 5 a m	57 10				
8– 9 p m	58 00				
3– 4 a m	60 65				
11–12 p m	63 22				
9–10 p m	72 94				
10–11 p m	73 54				

month with which we are concerned in precision, however, but the dispersion of volume of traffic among all the hours of the month April has frequently been considered a month near the mean in average volume of traffic, but in dispersion it is seventh for Holland Tunnel and eleventh for George Washington Bridge among the months (Table 6) October has been a favorite for traffic surveys, but that month is tenth for Holland Tunnel and twelfth for George Washington Bridge in dispersion of hourly traffic September is fifth for Holland Tunnel and ninth for George Washington Bridge

Surprising as it may seem the extremely heavy traffic of the summer months has the smallest hourly dispersion, and it is from the summer months that samples can be drawn with the smallest error of estimate The very best months for sampling based on the average for Holland Tunnel and George Washington Bridge are July, August, June, and March in the order named (Table 7) On the contrary the worst months for precision of estimate are December, October, November, and April in the order named

If the facts shown in these seven tables had been known when the Holland Tunnel and George Washington Bridge were sampled, the precision of estimate could have been increased materially by eliminating from the day period the hours from 6 to 8 in the morning and 8 to 10 at night If that survey had been made in the field, the men could have been given a six-hour day almost all in daylight and the precision of estimate would have been increased

But among the days of the week and the months of the year, there is little choice We may, therefore, allow the field men the freedom of the whole year for their work program instead of con-

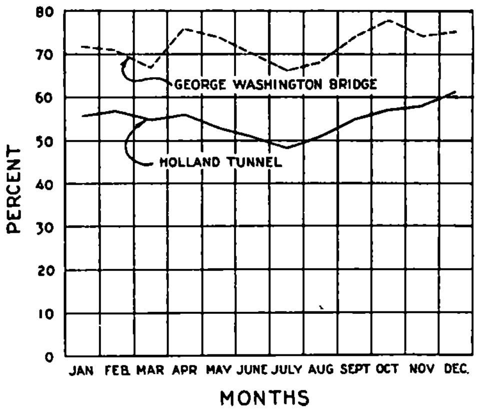

Figure 3 Dispersion of Number of Vehicles by Hours for Each Month of the Calendar Year 1933 in Chronological Order

fining them to certain months or certain days of the week If we do find it feasible or necessary to restrict a survey to a short period, however, it is clear that the summer months and March are preferable to the other months and that October to December and April are the worst

SEEING ON THE HIGHWAY

RECENT RESEARCHES ON LIGHTING REQUIREMENTS

By Kirk M Reid, *Illuminating Engineer*
Nela Park Engineering Department, General Electric Company

SYNOPSIS

Since 1930 there has been a 32 per cent increase in night traffic fatalities, whereas deaths during the daytime have decreased 4 per cent A substantial majority of night accidents on rural highways occurs on a very small percentage of the total highway mileage

Poor visibility has proved to be one of the most important single factors in the abnormal hazard at night Experience indicates that on urban streets and on the more dangerous portions of highways, improved illumination has reduced night accidents one-third to one-half, at a cost less than one-third of the losses from the accidents prevented by the lighting The available data suggest the desirability of obtaining larger scale experience for the more complete analysis of the effect of lighting upon night accidents and traffic volume on heavily traveled and hazardous highways

Extensive researches have for the first time given us the instruments and technique for measuring the end-product, visibility, as affected by color of light, glare, light distribution, pavement brightness, and obstacle brightness Startling advances have come about in light sources and their control to provide the maximum seeing and safety per dollar of total cost

On a very high percentage of the highway mileage the lower traffic density and accident experience indicate that fixed lighting would be uneconomic On these roads more adequate motor car headlights and their proper use will have to be depended upon for better visibility at night New data have recently been added to the available knowledge of the relation between visibility distance and headlamp beam candlepower under typical driving conditions Measurements have been made of the effect upon visibility distance of variations in speed, glare, and reflection factor of obstacles Of particular significance are data obtained with observers unaware of the fact that they were engaged in a test, providing for the first time quantitative information on the effect of the driver-attention factor

The alarming growth in traffic fatalities is an after-dark increment Analysis recently made by the National Safety Council reveals that since 1930 night fatalities throughout the United States have increased 32 per cent as compared with a 4 per cent decrease in day fatalities At night, with only about one-quarter of the total 24-hour traffic on the road, 60 per cent of the traffic fatalities take place This means that the average fatality rate per vehicle mile at night is well over four times that by day On the more hazardous stretches of highway the differential is much greater The urgent problem, then, is to reduce the abnormal and rapidly increasing night hazard

Information on the causes of traffic accidents is becoming more specific Poor visibility has proved to be one of the most important single factors in the abnormal proportion of night accidents Other factors, such as fatigue, drinking, carelessness, all play an appreciable part in the greater hazard at night, but darkness—lack of adequate visibility—is the major difference in the night condition Good visibility mitigates even the effects of the other factors named

This conclusion is based upon extensive experience on urban streets and main highways where variations in illumination—and visibility—have permitted studies of the effect of this factor On these streets and highways, analyses of accidents in relation to the visibility provided have been made by insurance companies, by safety organizations, and by city and state officials

Many of these analyses have been presented at your annual meetings or have been published in your "Highway Research Abstracts" For example, Schrenk (1)* reported a doubling of the ratio of night to day fatalities when the Detroit thoroughfare lighting was cut one-third, followed later by a proportionate decrease in night accidents when the lighting was again improved Schrenk has just issued his latest findings (2) on 31 miles of Detroit thoroughfares where through modernized illumination the night fatalities over an eight-month interval have been reduced to one-fifth of the former rate Specifically, this means that there are now some 28 people who are alive and going about their daily pursuits who, by all reasonable computation, would be dead if the visibility had not been improved on those Detroit streets This striking reduction in fatalities—at the rate of some 40 lives a year on only a limited mileage of Detroit's thoroughfares—is something to give pause to anyone who may doubt the primary effectiveness of good seeing in safeguarding traffic

Vey has reported experience (3) on main New Jersey highways where the night accident rate on lighted sections was only one-third that on adjacent unlighted sections Other analyses have been published by such organizations as the Travelers Insurance Company (4) and the National Bureau of Casualty and Surety Underwriters (5, 6)

These studies have shown that one-third, one-half, and often more of the night accidents are prevented where reasonably good visibility is provided On urban streets and on those portions of highways with high night-accident experience, the cost of providing good visibility by means of modern street and highway lighting has been found to be less than one-third of the losses from the accidents prevented by the lighting (4, 5, 6)

* Figures in parentheses refer to list of references at end

Mr T H. MacDonald, Chief of the Bureau of Public Roads, has pointed out the importance of developing the revenues derived from the "earning roads" If good illumination—through greater safety and comfort—increases the traffic on a main highway only a small percentage, the taxes on the increased use of gasoline will pay a substantial part or all of the lighting cost Where the increased revenue exceeds the lighting cost—a reasonable expectation on many highways—there is a net cash profit, in addition to the above-mentioned dividends accruing through conservation of life, limb, and property. These considerations suggest the desirability of obtaining larger scale experience for the more complete analysis of the effect of fixed lighting upon night accidents and traffic flow on heavily traveled and hazardous highways

The problem of providing safe seeing on streets and highways is being attacked on two fronts—fixed street and highway lighting, and automobile headlighting The latter will, of course, always have to be depended upon for by far the greater part of the road mileage On city streets, due weight should be given to the requirements of pedestrians not only from the point of view of safety but also from the point of view of convenience and comfort in the use of the public ways

FACTORS OF SAFETY IN SEEING

In the development of both methods of lighting, knowledge is first of all needed of the illumination required for safe seeing And then we need to know something about requisite factors of safety to insure quick, certain seeing while driving Factors of safety are incorporated in the design of all other parts of the car and highway. Unfortunately, few people have had a sufficient knowledge of the science of seeing to appreciate the elements that make factors of safety in visibility imperative under actual conditions of driving at night.

A factor of safety of 1 in visibility might be defined as the illumination which, under favorable conditions of road and weather, enables an observer—stationary, with normal eyesight, and at fixed attention—barely to see, at a safe distance away, an obstacle which he knows is there. Contrast these ideal conditions with those actually obtaining in seeing on a highway. In the first place

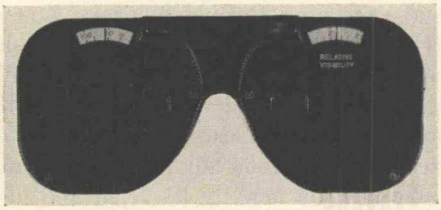

Figure 1. The Luckiesh-Moss Visibility Meter permits for the first time quantitative measurements of the end-product sought-visibility.

it takes time to see. Therefore with the car in motion one first sees an obstacle at a shorter distance than when standing still. Driving a car, it has been found, is not an automatic operation; it uses up part of the driver's sense capacity and thereby reduces his ability to see. As the speed increases, more sense capacity is used up in driving and the less he can see—and also the time element in seeing becomes more important. External factors—engine noise, radio, conversation, scenery—reduce the distance at which hazards are recognized. So do internal factors, such as preoccupation, inattention, and fatigue. The driver's eyesight may not be normal. Weather and road conditions may not be favorable. Nearly always an obstacle on the highway is not expected; it takes one by surprise.

Because seeing is such a complex physiological and psychological function, any considerable body of quantitative information directly applicable to the problem of safe driving has only recently become available. There had been a lack of instruments and techniques adequate for such quantitative measurements. Fortunately we now have them and are getting really significant data. Reference will be made later to some of these studies.

RESEARCHES IN HIGHWAY LIGHTING

Of the night accidents on rural highways, samplings show that a substantial majority occurs on a very small percentage of the total highway mileage. On these main highways, as on city thoroughfares, the traffic conditions are such that fixed highway lighting offers the most practical and effective means of providing safe seeing.

The possibilities of this approach to traffic safety have until recently had little attention from highway and safety organizations. This is not surprising because it is only within the last few years that there has been an adequate scientific and technical basis for a rational development of lighting for main highways.

The new instruments and techniques for measuring the end-product, visibility—as affected by color of light, glare, light distribution, pavement brightness, and obstacle brightness are themselves products of extensive researches. For example, the Luckiesh-Moss Visibility Meter (Fig. 1) now makes possible quantitative measurement of visibility. The Luckiesh-Taylor brightness meter (Fig. 2)—a telescopic photometer with highly restricted field—permits, for the first time, measurement of the brightness of limited areas of pavement at the grazing angles at which a roadway is viewed by a motorist. We are also using the Holst-Bouma meter, recently developed by the

Philips Laboratories in Holland, for measuring minimum perceptible brightness contrasts.

Recent years have made available new light sources, and marked improvements in former sources. Developments of new materials and new processes of fabrication have extended former limitations in light control and luminaire design. It was important to find out the relative merits of the new sources from the standpoint of safe seeing. It was important to find out how to utilize light on streets and highways so as to achieve greatest value to the public—that is, maximum seeing per dollar of cost.

So several years ago we undertook to attack this whole problem of highway seeing really fundamentally. Our first studies (Fig. 3) were conducted on an elaborate one-eighth scale test-model highway whose actual length was 250 ft. Through the flexibility and accuracy of this large scale model, researches (7) were completed in one year which would have required over ten years if conducted on actual streets.

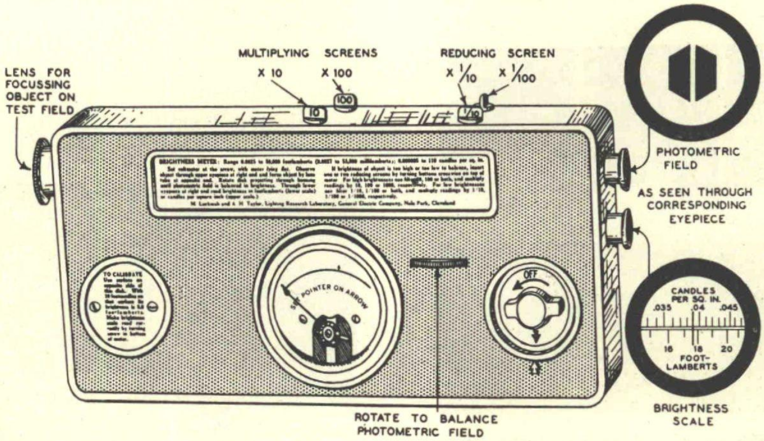

Figure 2. The Luckiesh-Taylor Brightness Meter has a magnified and restricted field ideal for measurement of the brightness of small pavement areas at grazing angles of viewing

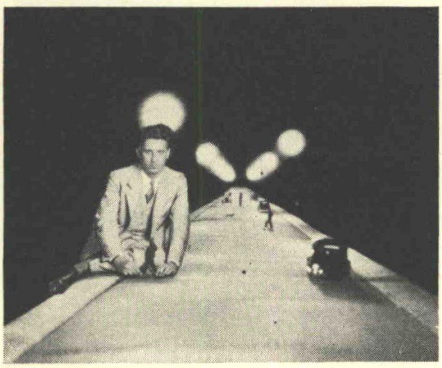

Figure 3. This realistic and flexible Laboratory Test Model Highway was employed in studies of fundamentals of street and highway illumination.

Measurements were made of visibility (Fig. 4) with lighting systems employing sodium-vapor, mercury-vapor, and incandescent lamps. It was found that for equal amounts of light, equally distributed, one cannot see obstacles any sooner or any farther away with one color of light as compared with others.

An exploration of methods of discernment (Fig. 5) revealed that at four out of five locations on a lighted highway obstacles are seen chiefly as silhouettes—that is, darker rather than lighter than the pavement background. Therefore, it is important to have substantial uniformity of pavement brightness, because one cannot silhouette a dark object against a dark area of pavement.

We found that glare from representative lighting systems may waste as much as half of the illumination provided, insofar as its seeing value is concerned. That is certainly not good engineering.

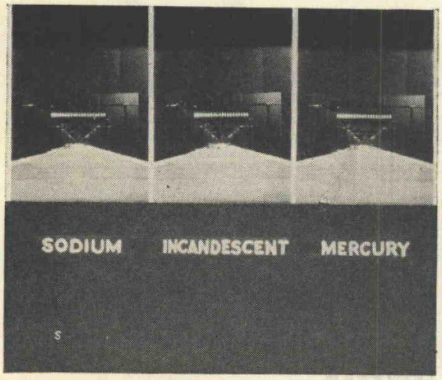

Figure 4. For equal amounts of light, equally distributed, these spectral qualities of light were found not to affect safe visibility.

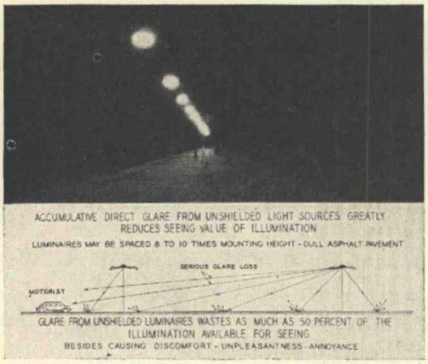

Figure 5. Glare from representative former lighting systems may waste half the illumination insofar as its seeing value is concerned.

"Spotty" lighting effects (Fig. 6) have been all too common. Contrary to popular impression, uniform illumination does not eliminate spottiness. This is what a uniformly lighted road looks like—to the driver of a car, not to an aviator. The model highway offered an opportunity (Fig. 7) for experiments in light distribution to produce the desired results of uniform pavement brightness.

Our researches have not been confined to the laboratory (Fig. 8). Measurements of visibility and related factors (8) have been made on streets and highways having various lighting systems.

Further studies, of a particularly comprehensive nature, are now nearing completion. Measurements are being made of the candlepower from luminaires vari-

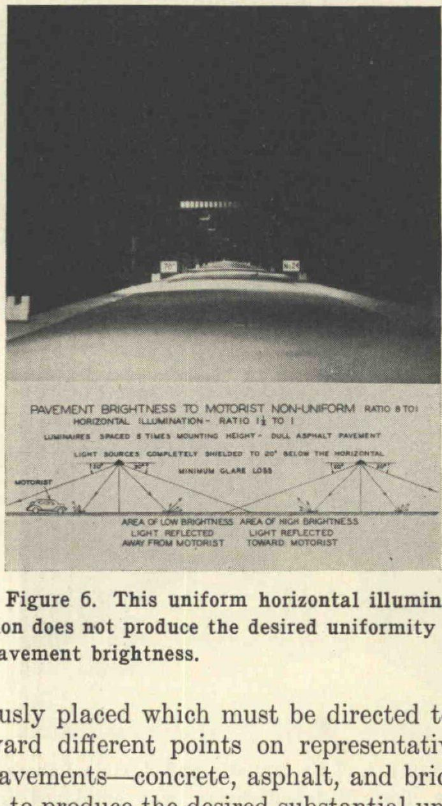

Figure 6. This uniform horizontal illumination does not produce the desired uniformity of pavement brightness.

ously placed which must be directed toward different points on representative pavements—concrete, asphalt, and brick—to produce the desired substantial uniformity of pavement brightness. Measurements are also being made of visibility of obstacle and pavement as affected by variations in pavement brightness, in obstacle brightness, and in glare.

These studies have influenced the design of modern luminaires for streets and highways. One by-product has been the development of the new High-Visibility luminaire (Fig. 9) for two and three-lane highways. It employs a new type of incandescent lamp, admirably

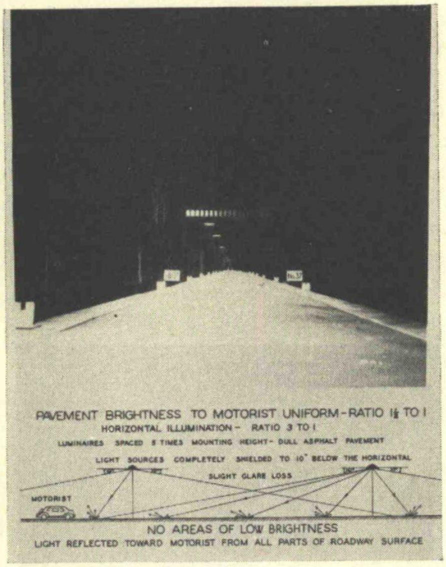

Figure 7. Obstacles are seen as silhouettes at 80 per cent of the locations on a lighted roadway; uniformity of pavement brightness is therefore of first importance.

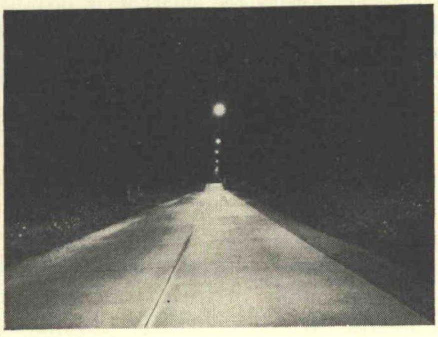

Figure 8. Measurements of visibility and related factors have been made on lighted streets and highways, as well as in the laboratory.

suited both mechanically and optically for road illumination. This lighting system (Fig. 10) has three outstanding features:

1. The proportion of light reaching the pavement is two to three times as great as that with former equipments.
2. Glare is suppressed by shielding the light source to approximately 10 degrees below the horizontal.
3. The road surface presents an altogether higher order of uniformity of brightness.

Through these gains in effectiveness, the seeing and safety obtained on the nar-

Figure 9. A direct result of researches in highway seeing is the new High-Visibility luminaire and improved incandescent lamp employed with it.

rower highways could have been equalled under past systems only by providing four or five times as much generated light.

The features of the sodium lamp (Fig. 11) were reported by Loewe (9) at the 1933 meeting. Because of its high efficiency of light production and the necessarily broad distribution of light from the large source, this lamp is particularly effective on the wider highways. It might be of interest that there are now over 5,000 of these in service in this country, for the most part on main highways and bridges. Two of the largest recent installations are on the Golden Gate Bridge and the San Francisco-Oakland Bay Bridge (Fig. 12).

Figure 10. High-Visibility incandescent lighting system near Albany. The seeing and safety provided by this type of lighting on 2- and 3-lane roads is greater than that from any other light source or lighting system now employed.

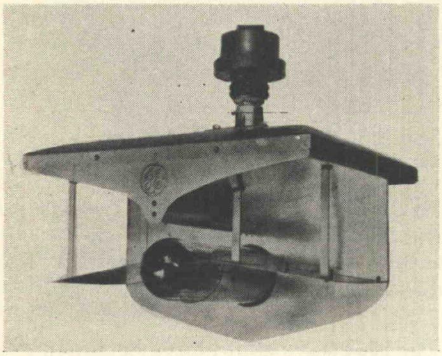

Figure 11. The new sodium-vapor lamp and luminaire. This lamp, because of its high luminous efficiency and its inherently broad distribution of light, is particularly effective on wide highways.

RESEARCHES IN AUTOMOBILE HEADLIGHTING

On a very high percentage of the highway mileage the lower traffic density and accident experience indicate that fixed highway lighting would be uneconomic. On these roads, as well as on many urban streets, more adequate motor car headlights and their proper use will have to be depended upon for better visibility at night.

New data have recently been added to the available knowledge of the relation between visibility distance and headlamp beam candlepower under typical driving conditions. Measurements have been made by Roper and Howard (10) of the effect upon visibility distance of variations in speed, glare, and reflection factor of obstacles.

Of particular significance are data (Fig. 13) obtained with observers un-

Figure 12. Among the outstanding installations of sodium lighting are the two new bridges at San Francisco.

aware of the fact that they were engaged in a test, providing for the first time quantitative information on the effect of the driver-attention factor. The data indicate that the average driver perceives the unexpected obstacle only half as far away as he sees the expected one. All of the drivers saw the unexpected obstacle at least 20 per cent as far as the expected obstacle, whereas none saw it at more than 80 per cent of the distance. Over two-thirds of the observers were grouped within a range of 40 to 60 per cent, and the over-all average is 51 per cent. The lower values indicate less than average attention, while the higher ratios indicate unusual attention or a chance fixing of the eye on the location of the obstacle at the critical time These data reveal that a seeing factor of safety of at least 2, and more logically 5, is required through variations in driver-attention alone Here is a guide to the practical adaptation of data obtained in the usual manner—that is, by observers who know they are taking part in a test of visibility distance

Figure 14 shows the relation, for several driving speeds, of beam candlepower to visibility distance for a dark obstacle Perception distance increases very rapidly for the first few thousand candlepower, but more slowly at the higher values The variation of visibility with candlepower as found in these tests is a straight-line relation between the logarithms of the two variables This conforms to the well established fact that the visual sensation—seeing—increases proportionately to the logarithm of the stimulus—illumination

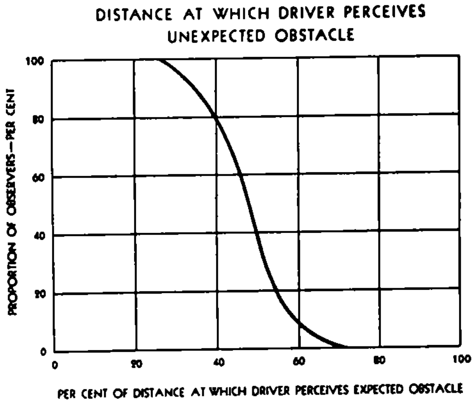

Figure 13 On the average, a driver perceives an unexpected obstacle only half as far away as he sees the expected one

These tests show a definite decrease in perception distance with increasing car speeds As would be expected, the spread among observers and among the determinations of a given observer was rather large But higher speeds always resulted in decreased visibility distance—for it does take time to see The over-all average shows a 20-ft loss in visibility distance for each increment of 10 miles per hour in speed, for observer-drivers not expecting the obstacle This loss, so far as these tests go, applies irrespective of beam candlepower and reflection values

Of course, in the case of light-colored objects, seeing distances increase materially Figure 15 shows the variation of visibility distance for obstacles of several reflection factors It is to be noted

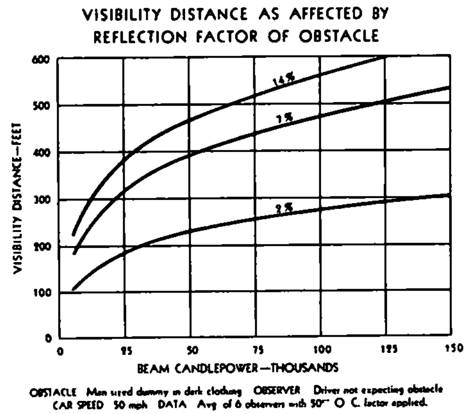

Figure 14 The distance at which a driver perceives an unexpected obstacle decreases about 20 feet for each increment of 10 miles per hour in speed

that visibility distance does not increase directly with reflectivity of the object, but more rapidly in the lower range than in the higher Changes in contrast or background and the angular size factor come in to affect the result

The type of road surface seems to have relatively little effect upon perception distance for an obstacle projecting above the road, though the road surface itself is visible for a greater distance ahead for the more diffuse, highly reflective surfaces such as concrete or crushed stone Under normal driving conditions, and with no light from behind the obstacle, the driver sees the unexpected obstacle in almost every instance by reflected light and contrasted with a darker background, even when the obstacle has a very low reflection factor and the road surface is light It was observed that when the driver knows that he will encounter an obstacle, he will frequently "pick up" the dark obstacle while it still appears darker than the background This change in the method of discernment accounts for greater spread in test results with dark obstacles, since the range of distance through which low contrast obtains is a considerable one

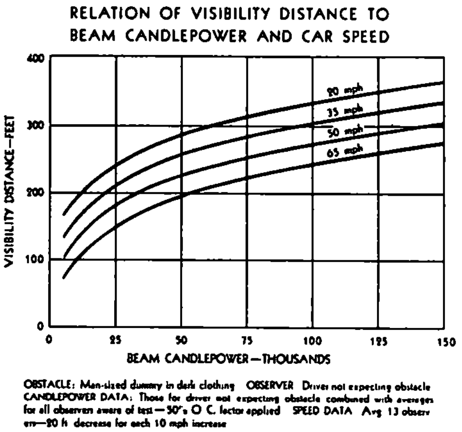

Figure 15 For a given beam candlepower, dark-colored obstacles are seen only about half as far away as light-colored obstacles

The tests confirm the conclusions of careful observers that, contrary to popular assumption, perception distances for obstacles projecting above the road are not reduced when the road surface becomes wet, even though the visibility of the road is radically reduced This is logical in the light of the test results reported in the above paragraph Furthermore, while wetting may in some instances reduce the reflectivity of the obstacle, the more specular wet road surface will reflect more light to the obstacle

Figure 16 presents the relation of perception distance to that required for stopping a car travelling at 50 mph Results are given for four values of beam candlepower The value of 20,000 beam candlepower is equal to or more than the average for headlamps found in service, 50,000 beam candlepower is the initial value for which most headlights have been designed in recent years, 75,000 beam candlepower is the maximum specified in the Uniform Code The stopping distance used in the chart is that published by the National Safety Council and applies to average driver reaction time, average brakes and tires, and to smooth dry pavements

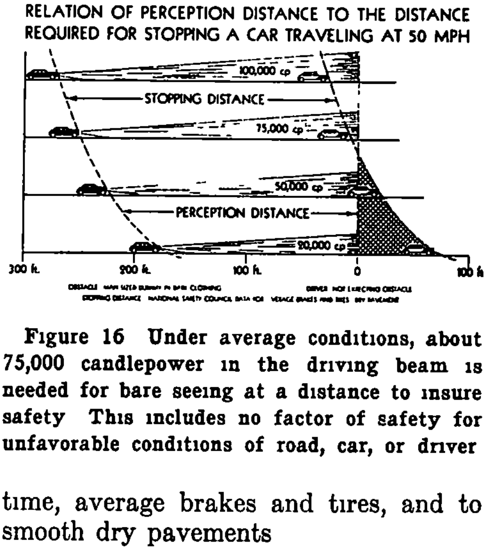

Figure 16 Under average conditions, about 75,000 candlepower in the driving beam is needed for bare seeing at a distance to insure safety This includes no factor of safety for unfavorable conditions of road, car, or driver

This chart shows that about 75,000 candlepower is needed, on the average, for bare seeing at a distance to insure safety It must be kept in mind that the data apply to average people, cars and road conditions There is no factor of safety whatsoever for impaired vision, older eyes, unfavorable atmosphere, abnormal distractions or inattention, fatigue, slippery roads, subnormal voltage, or brakes below par

It is well known that glare from approaching headlamps materially reduces visibility, and should be compensated for in the design of the meeting beam When an object is between the driver and approaching headlamps, it is usually fairly well revealed in silhouette against either the bright road surface or the headlamps themselves The most critical condition is that of an obstacle which moves onto the road after the approaching car has passed that location, in which case it must be discerned by reflected light In the Roper-Howard study, data applying to that condition are consistent with those reported by Moon and Warring (11) The average results of a series of tests with eight observers are shown in Figure 17 The candlepower of the headlamps illuminating the obstacle appears to have little effect upon the percentage by which glare reduces the visibility distance Roper and Howard expect to extend their testing in this aspect of the problem

It is apparent that visibility is reduced very rapidly for the first few hundred candlepower toward the eye, but walls off less rapidly as candlepower is further increased One thousand candlepower directed toward the driver is about the value from the well-adjusted lower or meeting beam of modern head-lamps It is seen to reduce the perception distance roughly one-third Seven thousand candlepower reduces the distance by about two-thirds

These glare data were obtained with a dark dummy placed behind the approaching car and at a lateral distance of 10 feet to the right of its headlamps If the obstacle is nearer the glare source, perception distances are still further reduced The location selected is, however, the most typical

You will have observed that the foregoing seeing data reveal the inadequacy of present headlighting for drivers observing moderate and legal speeds It is clear that under existing headlamp regulations motorists cannot always see with safety on unlighted streets or highways This fact accounts for the recent recommendation of the Eastern Conference of Motor Vehicle Administrators—already in effect in some states—that legal speed limits be lower at night than by day It is disturbing to consider what influence this might have (if enforced) in reducing the night traffic and therefore the revenue derived from the "earning roads", and also what effect it might have in diverting traffic from night to day, thereby adding to the present daytime congestion It is heartening, however, to

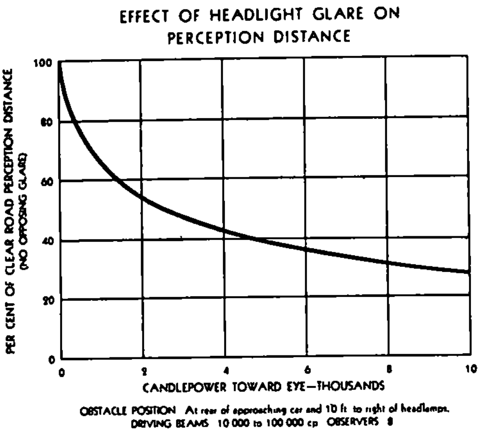

Figure 17. The decrease in perception distance due to glare from oncoming headlamps emphasizes the importance of using the depressed or meeting beams when meeting other cars

find the data indicating unmistakably that an adjustment in the terms of the headlamp regulations now imposed on the car manufacturers, undertaken in the light of the more complete knowledge, would permit them to make a very important contribution to the safety of night motoring

The car and headlamp manufacturers are now cooperating as a unit in the development of a superior system of headlighting They are coordinating their researches, and are taking other measures necessary to eliminate obstacles which might interfere with the broad program of providing safe lighting While still in its earlier stages, this program gives promise of an important contribution to traffic safety

In order to make a definite gain in safety there must be a radical forward step in headlighting The objectives of headlamp performance are clearly shown by the new knowledge of the requirements Driving beam candlepowers must be materially greater than at present, and the high-intensity portion of the beam must cover a wider spread The passing beam must direct very much more light at the right edge of the road, as Dickinson (12) and others have urged, it will then afford such obviously superior seeing when meeting other cars that the driver will choose to turn to it for his own protection as well as that of the approaching driver Means are being developed for more exact light control, which should be utilized in any new headlighting system instead of accepting compromises in beam patterns as in the past With the cooperation of all concerned it is possible to take the major step of providing headlighting which furnishes safe seeing initially and with maintained effectiveness

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DISCUSSION ON HIGHWAY LIGHTING

DR H C DICKINSON, *National Bureau of Standards* There is one point that I would emphasize—that is the need of more roadside illumination when one is using the medium or depressed beam We do not want bright roadside illumination when using the driving beam because that will detract from the visibility On the other hand, we do need roadside illumination when meeting other cars or making sharp turns

Such roadside illumination is incompatible with the means now used for going from one beam to another It can not be secured with two headlights which are alike—using two filaments in the same bulb and shifting from one filament to the other by a switch, that will require some other device We have not done anything experimentally By using two headlights which are not alike it is possible to get that kind of light beam However, that is not a very good solution

I hope in this program of research going on, that the companies will succeed in developing lights which will give adequate roadside illumination—that is, a spread of at least 100° when using the depressed beam and of at least 20 or 25° when using the driving beam.

CHAIRMAN MORRISON: I wonder if Mr. Reid will say something about street lighting. All the talks I have heard refer to rural highways. The streets of most cities are inefficiently and inadequately lighted. Is there any suggestion you can make about that?

MR. REID: The research findings reported in my paper have been embodied in modern lighting equipments for urban streets. These equipments are capable of providing adequate visibility for safety at moderate cost. The authoritative guide to proper installation practice is the Illuminating Engineering Society's Code of Street Lighting.

The Detroit experience is striking evidence of what can be accomplished with modern lighting equipments, properly applied. On many of the Detroit streets where the lighting was improved, there was no increase in the number or wattage of the lamps employed. The only change was replacement of obsolete by modern equipments, at adequate mounting heights and transverse positions. Yet the amount of light on the street was more than tripled by this change in equipment.

In many cases, of course, it is not possible to correct inadequate illumination so simply. Where the lamp size is too small or the mounting height is too low or the lamp spacing is too great, these basic deficiencies must be corrected in order to obtain safe visibility for night traffic.

The major street lighting problem, in my opinion, is no longer one of engineering. Efficient luminaires are available, and the requirements of proper installation are known. The problem is to arouse the public to a realization that adequate street illumination is an essential factor in night traffic safety, and that poor street lighting (with its preventable accidents and crime) really costs at least three times as much as good lighting.

Most municipal officials understand the value of good lighting and would like to provide it. But they are reluctant to spend the necessary money without assurance of the public approval.

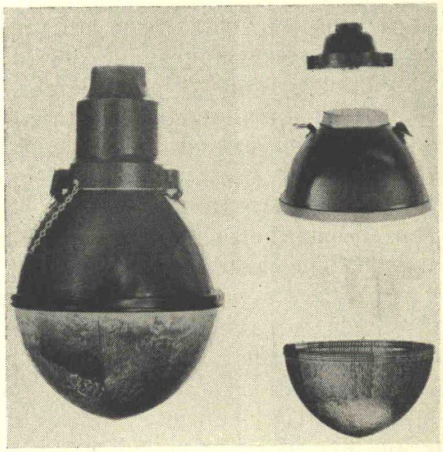


Figure 18. Research findings are embodied in this new and radically improved luminaire for street lighting.

Progressive officials—in Detroit, Syracuse, Akron, Troy, Hornell, Lynn, and other municipalities—have found an effective solution. They have selected those streets or portions of streets having the worst night-accident record and have lighted them adequately as "safety demonstrations". The first-hand observation and the invariable reduction in night hazard have crystallized public support of further lighting improvements needed.

CHAIRMAN MORRISON: I am thinking of an intersection where there have been a number of pedestrian fatalities and injuries. There were four light standards, one on each corner, and each had five lights. A hundred feet away in every direction there was another one. They were like so many Christmas trees—all kinds of illumination—but not on the pavement. Is not a boulevard system of lighting inefficient so far as safety is concerned? It is all right to make a pretty picture, but what about safety?

MR. REID: Professor Morrison asks about the five-lamp clusters. This is an ineffective and inefficient type of lighting equipment introduced during the early years of the century, when the 100-watt vacuum type was the largest practical incandescent lamp. The development of the gas-filled type of lamp, in 1913, permitted manufacture of lamps of greater light output and of much higher efficiency. Improved luminaire designs followed, with the result that the ball cluster became obsolete some twenty years ago. The cluster served to give the street a festive appearance, but it cannot provide safe visibility for modern conditions of traffic.

The illumination of intersections for pedestrian safety is an interesting problem, and one which is not as simple as it appears at first thought. It is logical, of course, to provide more light at intersections than between intersections, and the Code of Street Lighting so recommends. But if the street areas between intersections are poorly illuminated, even though the intersections themselves are quite well lighted, the pedestrian hazard at the intersections is usually found to be abnormally high.

The reason for this situation is that obstacles on a roadway are seen predominantly as silhouettes—that is, as darker than the pavement beyond. If the mid-block pavement areas are dark, this important silhouette method of discernment of obstacles at the intersections is largely lost. The conclusion is that for pedestrian protection the entire street length should be lighted adequately, with somewhat increased illumination at the intersections. It might be noted that the visibility requirements for vehicular safety are the same.

The sodium-vapor lamp has two features which suggest that it may prove particularly valuable for use at such points as railroad grade crossings and dangerous intersections, both rural and urban. First, the sodium lamp can be made successfully only in the larger sizes, which means that wherever it is used there will be a substantial amount of light. Second, the light from the sodium lamp has a characteristic yellow-orange color which one associates instinctively with caution. Installations now in service in several states—on railroad crossings and dangerous intersections—will probably permit evaluation of the practical effectiveness of these features.

DR. A. R. LAUER, *of Iowa State College*: I would like to ask several questions about highway lighting.

(1) How much light, measured in footcandles, is needed?

(2) How much does it cost to light a mile of highway?

(3) After driving under sodium lighting for an hour, would not the adaption of the eyes to the yellow-orange color destroy the value of the caution feature?

(4) Is not the monochromatic color of sodium light fatiguing to the eyes?

(5) Does not the greater cost of the sodium lamps fully offset the saving due to their lower wattage?

MR. REID: I shall try to answer Dr. Lauer's questions in the order asked.

(1) For pavements light in color, the Illuminating Engineering Society's Code of Highway Lighting calls for 0.2 to 0.5 lumens per square foot delivered on the pavement. These values correspond to 0.2 to 0.5 footcandles, measured on a horizontal plane at the street surface. For the darker pavements, the Code

recommends an increased amount of light

(2) The cost of lighting a highway varies rather widely with the conditions to be met Under average conditions, installation costs for a utilitarian system are usually in the range of $2,500 to $3,000 per mile; and operating costs, exclusive of investment charges, are usually in the range of $600 to $1,000 per mile per year

These costs are high enough to make it apparent that the place for lighting is on the heavily traveled and hazardous highways As stated in the paper, samplings have shown that a substantial majority of the rural night accidents occurs on a very small percentage of the total highway mileage On this limited mileage, studies by insurance organizations and highway departments indicate that the cost of good lighting is only about one-third the cost of the accidents prevented by the lighting

(3) After driving under sodium lighting continuously for an hour one's eyes would undoubtedly become so adapted that the light would appear to have lost its characteristic yellow-orange color to an appreciable extent This condition would not lessen the "caution" value of sodium for the lighting and marking of danger points along streets and highways where the great majority of the illumination is supplied by incandescent lamps

(4) Several tests have shown that monochromatic light has no disadvantage as regards eye-fatigue In one study, conducted by the Port of New York Authority, critical office work was performed for several weeks under sodium light, with no measurable effects upon the workers' eyes

(5) It is correct that the greater cost of the sodium lamps just about offsets the saving due to their lower wattage, for the average energy rates As production on the sodium lamp increases its cost will undoubtedly come down

For lighting highways the choice between illuminants depends largely upon the pavement width On wide roads the inherently broad distribution of light from the sodium lamp is particularly effective On the narrower roads the accurate control of light possible with the concentrated incandescent-lamp sources results in greater seeing per dollar of total cost On urban streets the color of the light from vapor sources is generally considered to limit their use to special conditions—such as dangerous intersections for sodium, and business areas for mercury in combination with incandescent lamps to modify the color.

PROF GREENSHIELDS · On city streets, the brightness and moving patterns of electric signs are often a handicap rather than a help to visibility for traffic safety In New York City, where the traffic signals are sometimes three blocks apart, confusion between traffic signals and colored signs is particularly serious What do you think is the best solution to this traffic signal problem?

MR REID Where a traffic signal is so located as to be viewed against an unfavorable background, its visibility can be improved by using a larger lamp or by putting an opaque shield a foot or more in width around the signal Good practice calls for signalization of each intersection where traffic control by signals is warranted

MR MAURICE HOLLAND, *Director of Engineering and Industrial Research, National Research Council* I would like to identify the major groups, social, economic or public, which are throwing hurdles in the way of what the highway engineer and the technician would like to do For example, where is the centre of economic interests in this whole job? Whether they like it or not, that would be the automobile industry itself At their last show the emphasis was on the technical subject of safety—having to do with the safety of steel body, safety glass, and things like that That is for the automobile population

Now about the poor old pedestrian The automobile industry, the Automobile Manufacturers Association, the Sloan Foundation, the insurance companies or some group who have a real interest in Mr and Mrs America walking around the streets, should give some encouragement and support, morally, politically or by pressure groups in municipalities, to put in force some of the recommendations which you have made

MR REID It is true that inadequate illumination takes its major toll, not among motorists, but among pedestrians on city streets and along main highways If the public will demonstrate a sincere desire for traffic safety, as you suggest, we can have it The requirements of safety are well known—proper judgment in driving and walking, rigid enforcement where needed, and provision of effective safety facilities (of which good lighting is one) The amount of expense involved is not great—undoubtedly less than the present accident cost But the public must show a real interest in safety

As to the lighting of heavily traveled and hazardous highways, there has been some opposition For the most part I believe the objections have come from groups which feared wholesale and unwarranted lighting of highways, or which have questioned whether the lighting of selected main highways is effective as a safety measure and economically sound Reliable data are limited, although more is known about the average cost of preventing an accident by lighting than by most of the other safety features of highways Larger scale experience is needed to settle any doubts of the value of properly applied highway lighting in reducing accidents and increasing traffic at night

MR V J BROWN, *Publishing Director, Roads and Streets* I have a lot of night driving to do and I notice one thing with respect to lighting that I think we are overlooking as highway engineers, and that is the wasted headlight value of our own cars on curves and on certain intersections As I drive around I have noticed that reflected light from certain types of roadside signs gives me as much value as the light of a street lamp, sometimes better, because the light so strikes the surface as to silhouette without the interference of the light source This reflected light can be gotten from our own headlights by the proper application of some kind of reflectors along the roadsides—take our own headlight beams and throw them back on to the road several hundred feet ahead of us

MR REID I am inclined to think the additional amount of light obtained on the pavement from such reflectors would be small Except where used in conjunction with guard rails, reflectors might interfere with shoulder development and maintenance Trial is being made in Michigan of a row of reflector buttons along a highway to mark the shoulder edge

ACCIDENT RECORDS AND TRAFFIC REGULATION

SOME IMPLICATIONS OF COST AND ORGANIZATION

By E W James

Chief, Division of Highway Transport, U S Bureau of Public Roads, Department of Agriculture

SYNOPSIS

There is a fairly developed consensus that the means for securing the greatest results in the direction of safer and more efficient highway operation include

(1) The control of licensing through training and examination,
(2) The inspection of motor vehicles and their proper conditioning,
(3) A detailed knowledge of when where and how accidents occur,
(4) A record of accidents so maintained as to facilitate the determination of proximate and contributory causes,
(5) A system of highway patrol,
(6) Methods of detecting violations of traffic regulations and of fair, reasonable and certain enforcement,
(7) A method for determining the driver having more than his share of accidents,
(8) A system of records suited to statistical analysis of essential facts relating to accidents

The development of these or other means considered necessary for securing the results desired is exemplified in similar auxiliary services already developed for the other recognized modes of transport—marine, rail and air

In connection with marine transport, there are the Coast and Geodetic Survey, the Hydrographic Office of the Navy Department, the Bureau of Lighthouses, the Bureau of Marine Inspection and Navigation, the Division of Marine Hospitals of the Public Health Service, the Coast Guard, and the North Atlantic Iceberg Patrol, and admiralty work in the Department of Justice

For railroad transportation, there are the block signal system, the interlocking and other safety devices on the permanent way, car and air-brake inspection, railroad telegraph and telephone systems a complicated financial and accounting service, and most of the Interstate Commerce Commission

And, for the air service, we find the same sort of services being developed, such as the established service of beacons and signals including radio direction control, a meteorological service and increasing requirements for more rigid inspection

Without undertaking to cover all the minor developments, we find the principal services represent large investments in capital plant and heavy annual operating costs, as follows

Maritime transport	Capital investment of auxiliary services $142 650,000, annual operation of such services $46,625,000
Rail transport	Capital investment, $480,000 000, annual operation, $62,400,000
Air transport	Capital investment, $34 400,000, annual operation, $1 740,000

In the field of highway transport, corresponding though not similar auxiliary services will probably develop and an approximation of their possible cost is indicated based on the present apparent tendencies

Assuming that present indications as to the prerequisites cited are sound, and that these will be eventually represented by more or less uniform adoption and organization throughout the United States, it appears that the costs may amount to the following

(1) For capital investment in the needed plant depending on extent of development, from $76 600,000 to $124,000,000
(2) For annual operating charges from $169,000,000 to $366,200,000

While we are at the moment considering principally the details of methods and means of securing greater safety and efficiency in highway transport as a utility for moving persons and goods, it is time we paused to study the implications of a general extension of the detailed operations that may finally be considered necessary and adequate

Recent studies made by the Bureau of Public Roads and the Highway Research Board in the general field of highway safety under a special authorization of Congress disclosed that many States are making an attack on the accident problem along similar lines, but in most cases the methods adopted are so differently administered that there is comparatively little cumulative advantage in the work being done because of its variety and incompleteness

We may agree, as most States do, that prerequisites of accident prevention are (1) The control of licensing through training and examination, (2) inspection of motor vehicles and their proper conditioning, (3) a detailed knowledge of when, where and how accidents occur, (4) a record so maintained as to facilitate use in determining proximate and contributory causes, assigning responsibility, indicating especially hazardous locations, conditions, or driving practices, (5) a system of highway patrol operating similarly to a rural police organization, but in addition capable of making an intelligent study of accident data, (6) methods of detecting violations of traffic regulations and of fair, reasonable and certain enforcement, (7) the determination of the habitual violator of traffic regulations and of the accident-prone driver, and (8) a system of statistical analysis that will disclose the true significance of accident facts Without considering further the customary present administrative details of the usual State motor vehicle department, such as licensing, registering, accounting, theft detection, et cetera, or the promotion of better and uniform legislation that falls to the lot of most agencies administering regulatory activities, we sense at once that the list of prerequisites recited is such as may develop into a very costly organization

Every State today recognizes the need for one or more of these services in connection with highway or motor vehicle administration, and several States have organized agencies to meet several of these prerequisites to a greater or lesser degree How far such agencies are to develop will depend upon the extent to which their partial application favorably affects the accident record Whether these activities toward accident control and reduction, now generally accepted, are expanded or not, we may expect to have corresponding services developed, because the safe and efficient growth of motor transport is a vital matter in the general field of transportation The annual accident record incident to motor transport and the consequent losses in life and property are clear evidence that highway transport today is neither as safe nor efficient as it should be, nor as it probably can be made If we turn to marine, rail, or air communication, we find that certain auxiliary services have grown with the development of each of these major forms of transport, and the number and magnitude of these activities are impressive

For example, along with the expansion of marine transport, which historically is man's earliest form of an artificial means of communication, we have as auxiliary services, the Coast and Geodetic Survey, the Hydrographic Office of the Navy Department, the Bureau of Lighthouses, the Bureau of Marine Inspection and Navigation, the Division of Marine Hospitals, the Coast Guard and the North Atlantic Iceberg Patrol, and admiralty work in the courts None of these agencies is needed for the mere operation of a ship, but each has been developed in an effort, which at times has had spectacular and tragic impulses, to make marine transportation of passengers and goods reasonably safe and efficient

Corresponding services are operated in connection with railroad transportation There is the organized car and air-brake inspection, the signal service, the railroad telephone and telegraph, the use of safety devices on the permanent way, a complicated financial and accounting service, and the greater part of the Interstate Commerce Commission which is devoted to railroads The railroads could operate trains without any of these auxiliary services, and did so for years until there developed a really hostile public sentiment But in the interests of safe and efficient operation, all of these appurtenant details have been added with great improvement to rail transport

The same process is going on in connection with air transport There is the established beacon and signal service including radio direction control, meteorological service, and an increasing set of requirements for safety devices and a more rigid inspection Air transport service, however, is comparatively new, but as it expands it is exemplifying the same course of development as the others with respect to appropriate adjuncts to promote safety and efficiency

There can be no doubt that in connection with highway transport corresponding auxiliary activities will be provided on an equivalent scale, and eventually become just as permanent and integral a part of this method of communication as these varied services are a part of the other accepted and established modes of transport The fact that many States are in more or less close agreement as to the prerequisities and have in many cases established the nucleus of some of the services that appear worth while is evidence of the inevitable tendency.

It will be interesting to examine for a moment the cost of some of these side lines to safe and efficient transportation and point out the sources of revenue for meeting the charges

Considering those auxiliary services associated with marine transport, we find first the Coast and Geodetic Survey with an operating plant costing, with ships and real estate, approximately $6,550,000, and an annual operating charge, averaged over the last five years, of $2,675,000 The Hydrographic Office of the Navy Department spends about $550,000 a year and has a relatively small amount of plant which I have neglected The Bureau of Lighthouses has a capital investment difficult to determine, but it seems to be approximately $50,400,000 and requires an average annual maintenance and operation charge of $9,500,000 The Bureau of Marine Inspection and Navigation, handling steamship and boiler inspection, has a small plant investment of $300,000, and an annual charge averaging $1,500,000 What we know as the Marine Hospital Service runs a plant costing $25,400,000, at an annual cost of $5,750,000 Originally the charges for marine hospital services were met from so called "hospital money" collected as port dues against ships, and today "light money" is still collected under certain conditions, although such receipts are no longer directly assigned to the upkeep of the lighthouse service

The next adjunct to be mentioned is the Coast Guard, with a total investment on land and sea of $60,000,000 and an average running cost of $26,650,000

These services, appurtenant to marine transportation, by no means exhaust the charges that accrue to that mode of transport, but they do cover the larger and more important items

We have in these services a total capital investment of approximately $142,-

650,000 and an annual operating charge of $46,625,000 These figures are not exhaustive

There is a considerable amount of admiralty business in our courts, but it cannot be differentiated as regards costs without extensive analysis There are the Federal Communications Commission, the U S Maritime Commission, Bureau of Customs, Bureau of Narcotics, Immigration Service, and the quarantine stations connected with the Public Health Service, some or all of which agencies may perform duties properly assignable as services auxiliary to marine transportation No attempt is made to segregate such activities or appraise them

Turning now to air transportation, we find a similar set of investment and operating costs, but so far on a much smaller scale There is a mapping service, installation of beacons, *et cetera*, and the capital costs to date approximate $34,400,000 and the average annual charges are $1,-740,000

When we consider the rail situation we find a somewhat different set-up, but the purpose and the results are essentially the same as in the cases of marine and air communication The fundamental difference is that in these modes of transport, practically all the auxiliary services we have been considering are public in their nature—that is, they are maintained and operated at public expense and by public officials The corresponding services in connection with railroads are installed and operated generally by the public utilities themselves as a direct charge against that form of transport

These services include first the use of block signal systems for control of train movements In some cases, these signals were introduced voluntarily, and are now compulsory on first class railroads by law Track circuits for safety signals and devices have generally been voluntary installations, as have also the introduction and use of railroad telegraph and telephone lines The creation by the railroads of special accounting and statistical sections to meet the requirements of the Interstate Commerce Commission has been under legal requirement, while car and brake inspection services have been largely voluntary

Considering first the automatic block signal system, we find the approximate cost on the mileage at present controlled is $190,000,000 The track circuits, interlocking and other safety controls represent a present cost of $177,000,000 Railroad telephone and telegraph installations on owned lines amount to $113,000,000 These auxiliary services represent in total $480,000,000

Annual operating charges are combined for the first two of these items in the only source of information immediately available For operating the block signals and all track circuits, interlockers, *et cetera*, in 1936, the annual expense was $41,250,000, for telegraph and telephone operation and maintenance cost, $13,750,000 Carrying on the accounting and valuation work incident to the requirements of the Interstate Commerce Commission has amounted to an annual average of $7,400,000 for a 20-year period Other probably large items cannot be easily ascertained These include charges for car, locomotive and brake inspection for safety purposes, track inspection, safety campaigns, *et cetera* The total of the annual charges recited is $62,400,000

The illustrations I have used are not for the purpose of comparing probable costs or means of adequately improving highway transport Obviously, the means will vary as they do among the other modes of communication and be appropriately developed to meet the particular requirements of highway transportation The railroads have no service which corresponds to the Coast Guard or the Division of Marine Hospitals Shipping has no service corresponding to the block system for control of train movements The prerequisites I have cited, which are at the moment accepted by many States as the most promising means of increasing the safety and efficiency of highway transport, have little resemblance to any of those means which are now in advanced development in marine, rail or air transport The cost of corresponding and appropriate auxiliary activities in highway transport may not be indicated by that of the services in the other modes of transport The cost may eventually be greater or less, depending entirely upon what line of growth is followed It will be interesting, however, to follow through some of the lines at present indicated for highway transport and see what the costs may be

Considering our first prerequisite of training and examination for drivers, we find some interesting proposals In Pennsylvania, a plan has been put forward for constructing eight training grounds with a road layout involving much the same variety of conditions met on public roads, such as, curves, sharp turns, narrow roads, and many types of intersections, stop, slow and caution signs, speed signs, and stop and go signals Arrangements are provided in the layout for parking in various ways, and others compelling backing and turning in both wide and narrow spaces There will be a small lecture room in which group instruction can be given and where the essential details of the vehicle mechanism will be taught to prospective drivers

Auxiliary to this field there is proposed a 5-mile section of road with similar variety of conditions including hills and vertical curves, long and short sight distances, *et cetera*, on which the drivers will receive final instructions in driving on the open road It is estimated that these installations will cost $2,000,000 If we assume that car registration is an index of the number of drivers to be inducted, and if the proposal for eight fields is sufficient and adequate for Pennsylvania, a corresponding development of similar training fields throughout the United States would cost approximately $29,000,000 In Chicago, a similar training field has been proposed and designed, and without the 5-mile section of open road is estimated to cost $3,000,000 If maintenance and operation costs are assumed to be ten per cent, the annual upkeep will amount to $3,000,000

In Connecticut, a scheme has been operated in connection with certain town schools, under which initial instruction and training have been given to promising pupils of suitable age If such a plan is followed through, it will mean expanded school facilities, both for equipment and additional teaching staff We will not venture an estimate of cost, but this item in any case would probably not be large

The second prerequisite of adequate vehicle inspection is one on which there is fundamental disagreement as to administration machinery An estimate made in New York for a sufficient and adequate State inspection service with stations and equipment was figured to involve a cost of $4,000,000 The same service plant for the United States would cost $47,600,000; and the inspection of 28,000,000 vehicles annually would cost at least $35,700,000, semi-annual inspections at least $70,000,000, and quarterly inspections $140,000,000 Inspections standardized in line with the questionnaire sent out under the auspices of Committee D-7 of the American Standards Association have been used in this rough approximation of costs, but it is questionable whether such short intervals as quarterly, or even semi-annually, are physically practicable I doubt also whether the estimate for equipment and plant made for New York contemplated a sufficiently large installation to handle 5,000,000 or 10,000,000 vehicles annually, which is the approximate number involved in semi-annual and quarterly inspections, respectively Perhaps the installation costs should be doubled or quadrupled

The third prerequisite of a detailed knowledge of when, where and how accidents occur ties in with the other requirement that an adequate highway patrol or police organization be developed At the present time, all but one State has some form of highway patrol or State police force organized or provided for On the basis of 24-hour service, the mileage covered indicates that one man is responsible for from 38 to 1,250 miles The corresponding figures, if present State controlled mileage only were covered, would be, 24 to 1,137 miles per man These lower figures indicate that in some States more than the so-called State roads are already covered by the patrol system We may carry this examination a step farther, to cover the latest available mileage figures for all rural roads We find in this case that the coverage ranges from one patrolman to 78 miles to one man to 28,700 miles This latter figure is, of course, fantastic, but there are at least 29 States where the coverage on this basis is one man to 1,000 miles or more This fact is an indication of the great expansion in organization necessary ultimately if all roads are covered

If we assume that the best present coverage appearing in the above figures represents the end to be sought in a highway patrol organization, we can estimate roughly the cost of such an agency covering the entire United States on the same basis If we provide for 24-hour service, using in the night shift one half as many men as during the day, that is, using 2½ men for the 24-hour patrol of each section, we shall need 31,000 men The average salary paid a patrolman from our record of 43 States is $137 per month Patrolmen alone would cost then $51,000,000 per year Most States allow expenses estimated at approximately $50 per man per month, or $18,600,000 Captains, Lieutenants, sergeants, *et cetera,* will add to this and at least 10 percent will have to be added for office supplies, equipment, overhead, *et cetera* This means about $78,000,000 for a uniform country-wide organization to cover the present patrolled mileage on a basis of one man to 38 miles of road

If we are of the opinion that 38 miles is too much for a single patrolman to cover, our figures will be raised In Pennsylvania there is a plan afoot to combine the State police and highway patrol, creating a force of about 1,600 men Even this force would on the basis assumed require each man to cover 58 miles, as against the best present coverage of 38 miles If we place the limit at 25 miles, the cost would be $118,500,000, and if developments required sections as short as 10 miles per man, the cost would be $296,250,000 per annum

This last figure is extravagant, but the special traffic officers in the District of Columbia average 33 miles per man, and if we consider the entire police force as active in traffic control as they in fact are when needed, the coverage is only 1 2 miles Somewhere between these two figures may be the desired one, and 25 miles to each man is not, therefore, so unreasonable Should we extend the coverage to all rural mileage as an ultimate goal on the basis of one man for 25 miles, the annual charge becomes $700,000,000 But no one expects that such intensive policing would ever become necessary

This is by no means the end of possibilities A proposal has been made seriously in Michigan to establish special accident squads covering territory so limited that they can operate practically as do the accident squads in such cities as Cleveland and Toledo, Ohio, and Wichita, Kansas Such squads would require at least two men each, and if they were to operate as indicated, their radius of operation would be not more than 15 miles, or about one squad to an average county The creation of such a force would probably permit a reduction in the regular patrol force, but this would by no means be so great as to offset the added cost of 83 such squads to cover the counties of Michigan Such an organization would cost at least $875,000 per year in that State A nation-wide organization would cost probably $26,000,000

With all the accumulated data that our field organization would collect, we should simply be swamped unless we had a trained and sufficient office force to handle it and determine what it all meant, and so we come to our accepted prerequisite of a properly maintained record and a system of statistical analysis to disclose the true significance of the information assembled What this organization would cost can only be guessed At present the annual cost of maintaining motor vehicle registration and licensing agencies, which also do such statistical work as is done, amounts at present to approximately $17,500,000 for 32 States The estimated cost for the United States is $26,250,000 We are now beyond the point of doing any estimating We can only guess what an adequate accident recording and analyzing office would cost Perhaps it would be twice or even three times the above figure, for the amount of material to be handled would be very great, and at the start the most expert statisticians would be required At least a part of the force would be high salaried and expert in nature

We need take no more time in presenting details There would be a great deal of court work and special courts may be necessary We have the illustration of admiralty cases before us as a part of the development around maritime transport Special traffic courts already exist in some jurisdictions These would have to be much more numerous and would sit most of the time

This whole presentation may appear fantastic to some But let us summarize the salient facts

(1) We know that the other commonly accepted forms of transportation of passengers and goods in the course of their development have required certain auxiliary services to make them reasonably safe and efficient

(2) These services have cost in plant, considering only the outstanding items, sums running into hundreds of millions of dollars In maritime transport, $142,650,000, in rail transport, $480,000,000, and in air transport, $34,400,000

(3) The annual operating charges of these agencies represent an aggregate of $110,765,000

(4) We are slowly coming to an agreement that certain corresponding auxiliary services are necessary for the safe and efficient operation of highway transport

(5) These agencies may cost tens of millions to install and hundreds of millions annually to operate

Assuming a uniform development on a minimum basis, the installation of training fields and inspection stations alone will cost approximately $76,600,000, and the annual cost of operating these, patrolling the highways, gathering accident statistics and analyzing them will amount to $169,000,000 annually If the organization is extended, the figures may be increased to $124,000,000 for plant and $366,200,000 for operation

The problem as it presents itself is First, to determine the particular forms which the auxiliary highway services are to take, second, to ascertain the extent to which these services will be organized and operated, and, third, to decide just how they will be paid for

These services will become a third grand division of highway costs, added to the present divisions of construction and maintenance Whether these costs are to be met directly from highway user revenues or indirectly from the highway users and others through general revenues will be an important question to be decided at a later time

DISCUSSION ON ACCIDENT RECORDS AND TRAFFIC REGULATION

MR L W McINTYRE, *American Motorists Association* I think Mr James hit the nail on the head, where are we going to find the money?

CHAIRMAN MORRISON We usually find the money for the things we want—we find the money for movies and we find the money for lipsticks

MR BURTON MARSH Mr James has done something that needed to be done and that is to face something that we must look at squarely and not sidle away from as we appear to be doing It is going to cost money, but if the public wants these things they are going to buy them

MR J S BURCH, *North Carolina State Highway and Public Works Commission* I think the Chairman was right a moment ago when he said we buy anything we want Our rather limited experience has demonstrated that so far the people do not want it, and if they do, they do not say so Individually a man is for safety, but at the same time it is very much like taking a drink, he wants his speed and he wants to have it in comfort and have everyone else get out of the way I think so far the practical answer to the question is that the people do not want it just yet

CHAIRMAN MORRISON I wonder if Mr James has reduced any of these figures to cost per passenger

MR JAMES That figure of 124,000,000 dollars for operation means $5 00 per car

CHAIRMAN MORRISON I mean on marine and air

MR JAMES I did not cover that point with respect to marine transport The figures I have given after all represent only the part paid by the United States for marine transport We take care of the marine transport of the world so far as it comes to our coast, and the other large maritime nations are also providing similar services If you wish to get the entire figure you must add these foreign services to the cost that I have set up here for marine transport You would not do so for railroads, of course, but for marine transport there are corresponding charges by the other countries which in large part should be added to these figures Because of our tremendous coast line, our marine services lap those of other nations and the reverse also is true

THE DETECTION OF ACCIDENT-PRONE DRIVERS

By Dr H M Johnson

Highway Research Board

SYNOPSIS

Study has been made of the accident histories of 29,531 Connecticut drivers, selected at random, each of whom has been licensed in that State for a period of six years It was found that reported accidents were not distributed among these drivers according to the laws of chance, and that the discrepancy between fact and chance-expectation cannot be attributed to chance

The drivers whose records were examined had an average of 1 accident among 25 drivers in one year, which does not mean the same as one accident per operator in 25 years The contribution of accident-repeaters to the total number of accidents was quite large, and far in excess of what the laws of chance permit In fact, a group of less than 4 per cent of the operators had 40 per cent of the fatal accidents, 36 per cent of the non-fatal personal-injury accidents, and 38 per cent of the accidents which involved no personal injuries

By dividing the accident experience into two periods, it was found that operators who were accident-free during either period had the lowest rate during the other, those who had one accident during one period had twice as many accidents during the other period as those who went accident-free during the first Those who had four accidents each during one period had between 7 and 9 times as many accidents during the other period as those who went accident-free during the first period

Drivers younger than 21 years had a disproportionate share of all classes of accidents Their fatal-accident rate was nearly twice the average, and their non-fatal rate about 1½ times the average A census of the whole driver-population of the State showed, with respect to fatal accidents, the same disproportions throughout each of five years, the critical age was about 21 If these relations are nation-wide we could have saved about 3,100 lives in 1937 by bringing the fatal-accident rate of persons under 21 to the average rate of their elders About 7,800 lives could have been saved by bringing down the rate of drivers under 25 to the average rate of those over 25

Thus two classes of drivers have been found who are now accident prone those who are now young, and those who have had a high rate in the past

Consider a given population of automobile drivers From it you are asked to select a class that will include the largest possible number of those who will have more than a prescribed number of accidents within a specified period, and the smallest possible number of those who will not You will call these selected drivers "accident-prone" They will then undergo some special treatment They may be discharged from employment, or transferred to non-driving duties, they may be lectured to, psychoanalyzed, sympathized with, they may be reproved, exhorted, rebuked, they may be forbidden to drive upon the public highways, they may be subjected to inconvenient restrictions, they may be refused insurance or be required to pay an excessively high premium It is therefore important whom you select

How shall you proceed? Two methods are now under consideration The first takes account of the drivers' present behavior, the second of their past Let us call them the examination-method and the biographical-method, respectively. We do not have to choose between them, according to their merits, we may adopt either, neither, or both

The examination-method takes two forms. direct and indirect The direct form requires you to take each person out to a standard course, which presents the same hazards to all Let the hazards correspond to those of actual traffic as nearly as possible Note how he deals with each of them and rate his performance accordingly Assume that those who are most inept in the test are also those who are most likely to cause trouble on the road This diagnosis presupposes, first that each person drives today in the test as he would drive today on the road, second that he drives on the road today as he will habitually drive during the period within which his accident-record is to be predicted To state these presuppositions is enough to cause them to be doubted The diagnosis depends on them

Consider now the examination-method in its indirect form Instead of having the driver operate a car under observation and under standard conditions, you subject him to a set of tests of particular skills, each of which is presupposed to be "necessary to safe driving", you subject him also to an interrogation about his "attitudes" or intentions Then, by combining his scores in the several tests in some manner which the testers have agreed upon, and by comparing the rating of each subject with that of every other, you formulate some rule, according to which you undertake to select those drivers who habitually drive the worst This procedure employs the same presuppositions as the direct method uses, and more besides If the rule of selection is valid, then there must be a usefully close association between the scores on the test and the accident-rates of the subjects Thus far, I have found no published report of a comparison which warrants the conclusion that test-performance is thus closely associated with accident-rate

In particular, the published reports of J -M Lahy, W Stern, Slocombe and Brakeman, Bingham, Sachs, and Tramm contain material which contradicts such a conclusion, even though some of these authors asserted it, while the published reports of Munsterberg, Miles and Vincent, C S Myers, and H M Vernon do not give the kind of evidence that is necessary to evaluate their conclusions that this is the case

In the United States, A R Lauer, at Iowa State College, and H R DeSilva and T W Forbes, at the Harvard Bureau for Street Traffic Research, have been developing different sets of tests which it is hoped will be related to accident rate Their reports published before this article was written do not enable one to conclude that they have yet succeeded, nor to conclude that they never will Our hopes, of course, are for their final success But in order to appraise the validity of any test or "battery" of tests, one must be able to compare the test-scores with some dependable criterion of driving performance which is independent of the test The best criterion, perhaps, is the operator's driving history—the duration of his experience, the amount and kind of his driving or his rate of encountering hazards, his accident-rate, etc To enable one to judge the tests requires records on some thousands of drivers; to collect and assemble the data ready for the statisticians is expensive, in one recent investigational series of tests about $6 50 per driver, the money has to be raised, and thus far there has been no rush of would-be contributors Hence, these investigators require time

Some attempts toward acceleration of these pre-appraisal activities have been made With the cordial and able cooperation of these investigators with the Highway Research Board, the U S Bureau of Public Roads, and the Commissioner of Motor Vehicles of the State of Connecticut, some 3,600 drivers were recently subjected to the Iowa State and the Harvard Bureau tests, in whole or in part The test-scores are now being correlated with one another, and also with the annual number of reported accidents, determined from the official records of the Department of Motor Vehicles This study cannot be reported until it is completed and the remaining work will require several months Meanwhile, the most that should be said is that the appraisal of the tests is no simple matter

Strong claims have been made for the validity of certain sets of aptitude-tests designed for taxicab drivers—some by A J Snow, and others by David Wechsler, and for street-car drivers by Morris Viteles Inquiries addressed to the presidents of the employing companies brought out that Wechsler's tests have been abandoned by the company which first sponsored them, that Viteles' tests are still in use and highly regarded by the personnel department, on grounds which the correspondent did not analyze The inquiry about Snow's tests has not been answered On the whole, one should strive to be open-minded and skeptical about the value of all indirect tests, being predisposed neither toward premature rejection nor toward precipitate acceptance of the claims that have been made for their merits

Consider now the biographical method Instead of trying to ascertain how each individual drives on a given date, and thence to infer how he habitually has driven, now drives, or will drive, one may try to find out how he habitually drove in the past This presupposes a reliable accident-history

Some psychologists assume that one can get this information from the driver, by asking him questions, on one date, about a long history They presuppose that he has good intentions, but also a good memory—that his recall of past events will not be affected, for example, by an inconvenient tendency to forget all that might upset his belief that he is an excellent driver But in many drivers this belief is practically a systematic delusion, on this subject they are paranoic A much better way is to choose a population whose accidents have to be reported in detail, very soon after they occur, the testimony of more than one person, and also of the objective circumstances, being recorded Some large fleet-operators record these data, few, apparently, have made good use of them Among those who record and use them are certain utility companies and certain departments of the United States Government

These records, if properly kept, serve to identify a class of drivers who have repeatedly had accidents in the past, and the operators who are thus classified tend very strongly to give predictable histories, not so much as individuals as classified groups

Can such records as a state obtains enable one to select a class of drivers whose group-performance can be reliably predicted? The question is important To answer it, we chose the state which kept its records in the form most convenient for our purposes namely, Connecticut From a list of about 408,000 drivers known to have been licensed in 1931, our workers selected every 10th name This gave a "random" selection of 40,800 names From this selected list all drivers were eliminated unless the records of the department showed that they were licensed in every year in the period 1931-1936 This left a population of 29,531 drivers, for whom the official reports were complete for these six years Among them there accrued 7,082 accidents, which involved 5,650 operators If the accidents had been distributed among the operators irrespective of their personal identities and histories, then the distribution could be described by Poisson's law of small chances, for which the sole parameter is the rate of accidents per operator, within limits of inaccuracy which are defined by the parameter itself If, within those limits, the law is not satisfied, then the distribution of accidents per operator is not impartial, but is subject to some systematic influences which remain to be determined experimentally

Table 1 shows that the actual distribution of these accidents does not follow the laws of chance Note that there is an excess of accident-free operators, and also an excess of repeaters, with a corresponding deficiency of operators having a moderate accident-rate We analyzed nine other populations in the same manner, and found that eight of them showed this characteristic Table 2, based on the driver-population studied by Slocombe and Brakeman, brings out the discrepancy in a more striking fashion These authors included all accidents, regardless of damage, and thereby obtained a much higher rate of accidents per operator than obtained in our population

Figure 1 shows what kind of predictions can be made from these histories Here we classify the operators according to the number of accidents which they had in 1931-1933, and show their accident-rates for 1934-36 Bear in mind that three years is a short time in which to establish an accident-history if we consider, as in this case, only those accidents which occur, in the average, once in 25 years of an operator's experience And yet, those who had one accident each in the first three years have twice as many accidents per operator in the second three years, as those who went through the first three years accident-free The probability is only $(10)^{-24}$ that the difference between these two rates is due to chance And those who had more than one accident in the first period have correspondingly higher rates in the second period

Figure 1 shows the converse relationship also Here we may see that not only can we predict the future performance of these classified groups from knowledge of their past performance, we can also infer their performance in the remote past from knowledge of what they have done in the less remote past In fact, the second kind of prediction

TABLE 1

Actual and Expected Distribution of Accidents, Including Casualties and Property Damage Exceeding $25, Reported to the Commissioner of Motor Vehicles of Connecticut, 1931-36, in a Licensed Driver Sample Selected at Random

Accidents per operator during experience	Operators having these accidents			Accidents accruing to these operators		
	Actual number	Expected number	Difference	Actual number	Expected number	Difference
0	23,881	23,234	647	0	0	0
1	4,503	5,572	−1,069	4,503	5,572	−1,069
2	936	668	268	1,872	1,336	536
3	160	53	107	480	159	321
4	33	4 (4–7)	47 (4–7)	132	15 (4–7)	212 (4–7)
5	14			70		
6	3			18		
7	1			7		
Totals	29,531	29,531	0	7,082	7,082	0

Note The probability that the differences between the actual and expected distributions is due to chance = 1 6 $(10)^{-161}$, which is insignificant

TABLE 2

ACTUAL AND EXPECTED DISTRIBUTION OF ALL ACCIDENTS REPORTED TO EMPLOYER, REGARDLESS OF PERSONAL AND PROPERTY DAMAGE, OF OPERATORS OF ELECTRIC RAILWAY CARS AND MOTOR BUSES, EMPLOYED CONTINUOUSLY THROUGH THE ONE-YEAR EXPERIENCE (1927)

REPORTED BY SLOCOMBE AND BRAKEMAN

Accidents per operator during experience	Operators having these accidents			Accidents accruing to these operators		
	Actual number	Expected number	Difference	Actual number	Expected number	Difference
0	217	101	116	0	0	0
1	326	315	11	326	315	11
2	569	493	76	1,138	986	152
3	458	514	− 56	1,374	1,542	−168
4	258	402	−144	1,032	1,608	−576
5	145	252	−107	725	1,260	−535
6	99	131	− 32	594	786	−192
7	86	59	27	602	413	189
8	46	23	23	368	184	184
9	27			243		
10	24			240		
11	24			264		
12	7			84		
13	5	10	86	65	103	935
14	2			28		
15	2			30		
16	2			32		
17	2			34		
18	1			18		
Total	2,300	2,300	0	7,197	7,197	0

Note The probability that the differences are due to chance = 4 $(10)^{-217}$

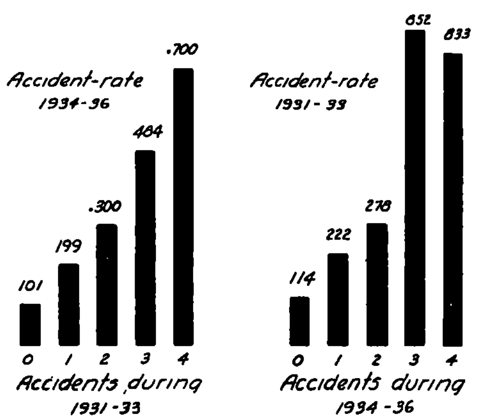


Figure 1 Relation of numbers of accidents in each half of the six year experience with the accident rates of the same groups in the other half

is more accurate than the first, perhaps because the younger drivers performed worse in their early experience, and so gave us more to predict

The accident-repeaters in this population were 3 88 per cent of the total Together they caused 39 per cent of the fatal accidents, 36 per cent of the non-fatal personal accidents, and 38 per cent of the non-personal accidents Thus, they have nearly the same proportion of every kind of accident For this reason the employer, the insurance company, and the licensing authority ought, if possible, to have a record of all accidents whether their consequences are important or trivial, in order to establish a reliable basis of prediction as early as possible

The accident-repeaters tend to shorten

the time between accidents as their accidents accumulate the fourth accident, for example, tends to follow the third more closely than the third follows the second

RELATION OF AGE TO ACCIDENT RATE

In this census the operators were classified according to their ages, which yielded some important information which is not evident in former compilations

Figure 2 shows that the drivers who were under 16-20 years old at the beginning of the experience and under 22-27 years old at its close had 1 47 times as many of the non-personal accidents as they would have had if the distribution of accidents were independent of age That this difference is not accidental is evidenced by the fact that the probability of the independency-hypothesis being true is less than $(10)^{-24}$ Figure 2 shows likewise that this same group had 1 53 times as many non-fatal personal injury accidents as the independency-hypothesis allows Moreover, this age-group had 1 83 times as many fatal accidents in this period as the independency-hypothesis allows Here the probability is as great as 1 in 250 that the hypothesis is true But, it is still a very conservative bet that the driver's accident-rate is influenced by his age That the odds are easier in the case of fatalities than in the other classes of accidents is due to the smaller number of individuals involved in fatal accidents

Thus, it looks as if those accidents which involve the youngest class of drivers are likely to be more severe than those which involve the older drivers

These findings led to a request for an age-census of all licensed drivers and all identified drivers involved in fatal accidents in Connecticut through the period 1932-36, that being the only period for which the individual accident-records had not been destroyed No state had made and published such a census up to that time The results are extremely valuable Figure 3 shows the age-distribution of the licensed drivers of Connecticut for 1929, and for each of the years 1932-36 Plainly, the average age is increasing, the proportion of youths becomes smaller in every year The probability that the populations of 1932

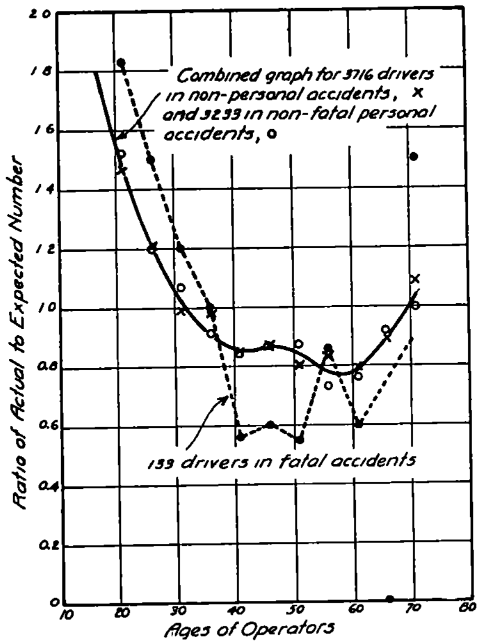


Figure 2 Relation of age to traffic accidents from histories of 29,531 operators selected at random from those licensed in Connecticut in every year from 1932 through 1936

and 1933 are homogeneous is only 1 in 250 in the next closest instance the probability is $(10)^{-24}$

Figure 4 shows the ages of 2,467 identified drivers who were involved in fatal accidents in Connecticut during the period 1932-1936 In this figure one year was chosen as the range of each age-class, in order to ascertain whether a critical age exists, especially in the youngest group Whether the data will bring out such a fact or not depends partly on the number of persons in each of the neighboring age-classes This condition is satisfied for the earliest ages, and it is manifest from the graph that a critical

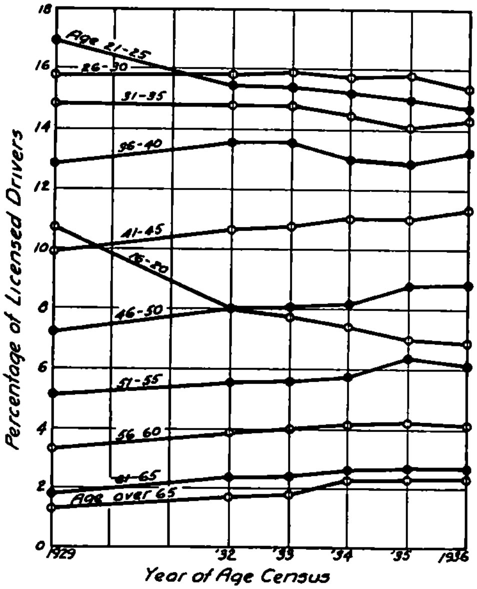


Figure 3 Relative number of drivers of various ages in Connecticut

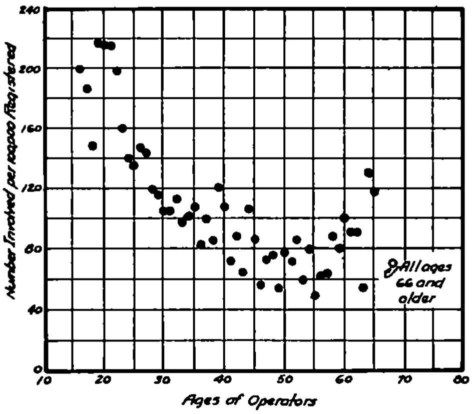


Figure 4 Drivers involved in fatal accidents per 100,000 drivers registered, Conn 1932–1936

period lies somewhere between the ages 19 and 21 Certain psychiatrists have suggested that the female population ought to show similar crises in the late 20's and the late 30's We have considered the two sexes separately, but since of the 2,467 identified drivers who were involved in fatal accidnts, only 136 were women—a little more than one-half of one per cent—and since the number of all drivers is by no means large for the purpose, it is not easy to make out any age-trends among the women, if such trends exist

Because the population is still rather small, we have grouped its members into 5-year age-classes, and have expressed the actual number of fatal accidents in each of these classes as a ratio to the corresponding number to be expected on the hypothesis that the rate per operator in each age-group is the rate for the population as a whole The black circles in Figure 5, and the curve fitted to them, indicate the result Thus, the age-group 16-20 years contains 1 73 times as many fatal-accident drivers as the hypothesis implies, the 21-25 year group contains 1 48 times as many, and so through The groups which contain the smallest proportion of involved drivers are 46-50 and 51-55 years The probability that these discrepancies between fact and hypothesis are jointly due to chance is $4(10)^{-48}$, which indicates practical certainty that the differences are statistically reliable

Disproportions of this same order appeared in every year of the experience taken singly, they also appear in three experiences from Massachusetts —in which, however, the basis of expectation is less reliable than in the present instance

The open circles in Figure 5 show the ratios between the actual and expected numbers of drivers suspended for speeding in the District of Columbia in the first five months of 1936, according to their ages This does not prove that these youths who have fatal accidents are the same youths who drive at excessively high speeds, but it does suggest that the manner in which these youths drive may deserve looking into

In 1936 the youngest age-group had 2 10 times as many fatal accidents per 1,000 drivers as the whole population had, and 2 26 times as many per 1,000 drivers as their elders had This ratio is the highest that appeared in any of these years But there is no evidence that their rate is increasing, or diminishing, except according to chance The rates for the older drivers, however, declined in every year of the experience, in the average by about four per cent of the rate for the preceding year Thus, the youngest drivers are growing relatively worse, because their elders are driving better or driving less

If the age-relations among the drivers in Connecticut hold on a nation-wide scale, it would have been possible to save the lives of 3,085 persons who were killed on the highways in the United States in 1936, if by any means whatever, we could have brought the fatal-accident rate of the drivers under 21 years of age to the rate of their elders, likewise, it would have been possible to save about 7,787 of the 37,800 who were killed, if we had reduced the rate of the drivers under 25 to that of their elders

These findings point to the weakest of all spots in safety-education and in administration According to all tests of separate skills which are believed to be used in handling a motor vehicle, the highest average scores usually belong to the age-groups in which we find the highest rates of fatal accidents and also of personal injury accidents Moreover, it seems to be true that most drivers 20-22 years of age are able to handle a car on the road more skillfully than their elders It therefore seems that the question is not so much how skillfully a person can drive, as whether he will use the skills that he has Some skillful drivers, relying on their agility and alertness, may enter hazardous situations that are a little beyond their ability, whereas less skillful drivers, being aware of their weaknesses, may stay out of them The fact that the youths are the ones who have been most often and most intensively preached to in recent years ought

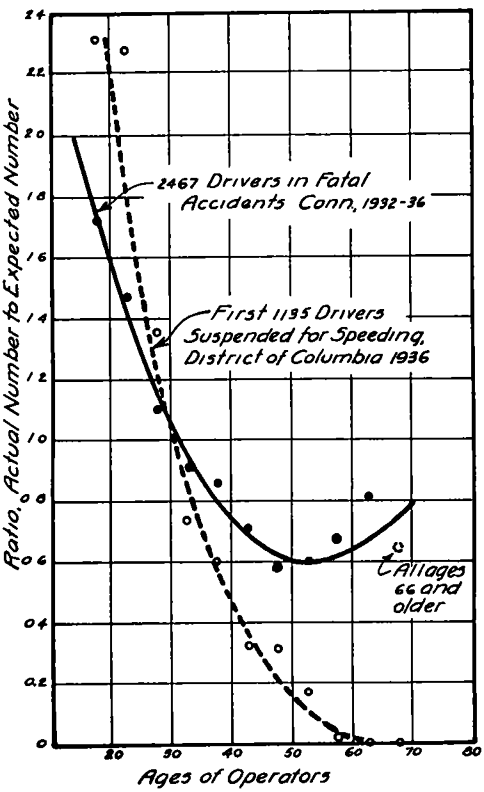


Figure 5 Comparison of ages of motor-vehicle drivers suspended for speeding in the District of Columbia with ages of drivers involved in fatal accidents in Connecticut

to suggest that our efforts in safety-education may be improvable

To summarize our biographical investigation has identified two classes of drivers who are now accident-prone They are, namely, those who are now young, and those who have had many accidents in the past

DISCUSSION ON ACCIDENT-PRONE DRIVERS

DR A R LAUER, *Iowa State College* There are a few points in Dr Johnson's paper on which I think it might be well to elucidate further relative to the methods involved Dr Johnson has presented the two methods, indirect and direct The biographical method would be all right if we had unlimited time to determine who should and who should not have a license We have had thrust upon us this problem which one cannot solve as has been suggested

I have never gone on record as supporting any of these tests Having analyzed several sets of similar data before, I venture a guess that when our final analysis is made we will find no high validity coefficients This is to be expected from the conditions and the phenomena with which we are working Human nature is unstable and highway conditions are highly variable Yesterday we heard talks on studies made of the maintenance costs of highways in different parts of the country As you remember, those maintenance figures varied from $100 to $500 or $600 a unit on different types of highways which apparently were comparable This is to be expected For similar reasons validity coefficients will never be high as long as we lump accidents into a unitary criterion They will not be high even if we segregate our accidents according to defective manipulation, defective vision, or how ever you may care to analyze them One reason for this is that there is some evidence that most accidents are caused by more than one condition In any one accident you may have three or four directly significant factors to consider We might mention the test for glare and accidents allegedly caused by glare The fact that one was blinded by glare would not mean necessarily that it caused the accident, but it is simply a factor which may or may not operate One may meet a thousand animals on the highways at night at such times that oncoming headlights are less blinding One will never get high validity coefficients so long as numerous factors produce the same end

Another point is this I have for many years maintained that the best test of a driver is 50 years of driving This would be the ideal test The best test of highway surface wear would be to lay the highway down and see how long it will last In the meantime your motor vehicle departments and others are faced with the practical problem of trying to keep certain groups of drivers from having so many accidents We can't wait until each one kills a few people to control his driving habits

I had lunch today with a man who lost his son He said go ahead and do what you can to reduce accidents but it is too late for me as I have lost my son If it is necessary to wait until a driver kills several people we certainly are not using the most economical method of dealing with the problem. What would we do with these young drivers? We cannot eliminate them from the highway for the simple reason that it is unconstitutional A remedy which might have some merits as practicable and which we will leave with you is that of legislation to reduce the speeds of certain persons or restrict their driving as some states have done at the present time Let a man drive according to his limitations

Now I have no brief to hold for tests as given by myself or anyone else As a matter of fact we have been changing our equipment constantly and you will find our clinic quite different than it was last fall We eliminate as soon as we find the test has no practical value However, as long as we are giving examinations and drivers are being charged for them at the rate of $5 or so per examination, as some states are doing, why not do the best job we can?

We should be interested in this problem from the standpoint of trying to protect persons from injury rather than to dictate to drivers whether they can or cannot drive The medical profession do not guarantee to cure any ailment, they merely examine a person, tell him what his physical characteristics are, and with their experience, may be able to tell, within reasonable limits, what his chances are of living for a certain number of years There is never certainty, it is a matter of probability The insurance companies have been building up a rather thriving business based upon prognostic examinations of this type In a similar way I fully believe a scientific examination will help drivers stay out of trouble

The final point concerns the matter of expense In the present instance we have been given a big problem and it is necessary to be rather careful in everything we do It is not exactly typical of the expenses necessarily incurred from regularly examining drivers in large groups We examined 2,500 persons in the State of Iowa at a cost of little more than 60 cents per driver A year before we examined 1,000 commercial drivers covering 4,000 miles for $1 50 apiece The cost depends somewhat upon the number of people you are examining and the economical use of the equipment One should try to be open-minded I should say that we should ask people who offer any test for sale or rent, regardless of what it is, to show that it does better and is more economical than what has been used before If it does better than what has been used before, it is justified, if not, there is no reason to change This is a simple and unbiased answer as to the value of any licensing procedure

Mr C M Johnston, *Bureau of Public Roads* I should like to ask Dr Johnson if both people in an accident were counted in tabulating the statistics

Dr Johnson These were involved drivers taken without regard to the distribution of responsibility I think most administrative authorities agree that you cannot distribute responsibility It is very seldom a man is involved in an accident except by doing something or failing to do something that would have kept him out of it

Mr C M Johnston, *Bureau of Public Roads* What I am trying to get at is, if you add the one who is 16 to 25 involved in the accident—he might have run into someone in the 40-year old group?

Dr Johnson Then you have two involved, each one being counted

Mr C M Johnston, *Bureau of Public Roads* If you can take that accident from the 40-year group and leave it in the 16-year old group, then how would the picture look?

Dr Johnson I do not know, that would require going back of the records that are available You would have to go into the analysis of the particular accidents which would take an expert to do and the facts are not available The number of deaths is about 0 97 times the number of involved drivers There are some few accidents that involve more than one driver and a few accidents involve more than one death for a given driver, but the ratio is very close to 1 to 1,—one accident, to one identified driver

Dr Lauer I don't beleve these data shaped up as well as they deserved You did not have mileage on any of them

Dr Johnson Not on any of them In another study of course we are recording some data that you obtained from nearly 3,000 drivers, namely their own reports of miles traveled I think that you will get an interesting age distribution You have already got that for that matter in your own work, but I don't believe I trust the reliability of those reports In the first place, if a man does not keep a record of his gasoline or of his speedometer, he does not know how many miles he drives in a year In the second place, he answers questions under circumstances that may invite him to exaggerate or to suppress and in the third place, everybody knows that a driver's hazards do not depend simply on his mileage In the fourth place, a very large portion of his hazards are hazards of his own creation

Dr Lauer There is of course no doubt that it is reasonable to suppose that a salesman 30 or 40 years old who travels daily travels more miles per year than a youngster

Dr Johnson Increased exposure may have something to do with increased rate, but I doubt very much if you can plausibly assume that children under 21 have a higher exposure rate than the average 30-35 year old group

Dr Lauer I was assuming they had less

Dr Johnson Then the youngsters are relatively worse than they now appear to be

Professor Morrison To me there is no mystery as to why young drivers have many accidents, and I doubt if it has much to do with their driving experience, in years or miles Even casual observation shows that when a car shoots ahead in traffic the chances are at least ten to one that the driver is a young person These young drivers are usually skillful but they do not have, or at least do not exercise, judgment Youth has always been impetuous, daring, and rather careless of consequences Also, it is usually young people, rather than their elders, who go out for a good time at night, visit roadhouses and have enough drinks to cause accidents

The general assumption is that in all traffic accidents it is the drivers who are to blame, but I believe that in the majority of pedestrian accidents the primary fault lies with the pedestrians In one city half the pedestrians killed in 1936 were either crossing streets against red signals or crossing between intersections, usually in the dark It takes a very careful and alert driver to avoid hitting an old man in dark clothes crossing in the middle of a block on a poorly lighted street at night, and hardly anyone can avoid hitting a child who suddenly darts out in front of him from between parked cars

THE HIGHWAY SAFETY PROBLEM

By R W Crum

Director, Highway Research Board

SYNOPSIS

It is apparent that there is no panacea for the hazards that have grown up along with the development of highway transportation No single corrective measure can solve the problem of highway safety

Recent research indicates the presence in the driving population of three classes of drivers a considerable group of accident free individuals, a relatively small group of accident prone or high accident individuals and a large number of drivers who participate in a large number of accidents more or less according to the laws of chance

While a first objective should be to segregate the accident prone operators and either eliminate them or change their habits by educational methods, it is evident that to reduce highway accidents to the lowest possible terms a long pull must be directed toward reduction of the total number of mishaps suffered by the great number of drivers who may be considered normal

To do this, it is equally apparent that sustained effort directed against many accident causative factors must be put forth The roads and vehicles must be built and maintained in the safest possible condition for normal use, uniform rules and control methods must be adopted throughout the nation, and the driving public must in some way be brought to adopt better driving habits and a better attitude toward the use of motor vehicles Owing to the statistical fact that the average driver's expectancy of being in a serious accident is only about one in twenty years, there is an indifference on the part of individuals that must be overcome Two factors which would repay immediate attention are the disproportionate share of highway accidents involving young drivers and the high rate during the hours of darkness

While the objectives can be clearly seen and the principal lines of approach are to some extent indicated, much more information on many phases is needed to make the attack all along the line effective The hazards of the road must be studied in relation to driving practice and motorists' behavior under all conditions must be exhaustively studied before the characteristics of good driving behavior can be authoritatively stated Definite information on the relation of intoxicants to highway hazards must be secured

Much money and well directed effort have already been expended in combating the hazards of highway travel Although substantial reduction in the ratio of accidents to traffic as measured by gasoline consumption has been effected, still with increasing travel the total of mishaps is growing It is generally recognized that something more is needed But what?

To devise corrective measures logically, knowledge of the facts is required Before any problem can be solved it must be broken down into its component parts and the available pertinent facts determined

During the past year a large amount of work and study has been carried on jointly by the Bureau of Public Roads and Highway Research Board in cooperation with many other organizations for the purpose of collecting definite information about some factors which are suspected of being near the core of the situation From study and observation during the progress of this work and study of the data secured, I have developed a picture of the situation that has clarified the matter for me Although nothing startling or revolutionary can be said the analysis of the facts so far as they are known brings into sharper focus some of the things that can be done now, and some upon which further light must be shed

The picture in broad outline is simply

this Only a minor part of the accidents can be assigned to definite groups in the population that can be identified as such The majority of the accidents happen to the great bulk of the drivers who cannot be segregated into independent groups This means that although definite measures are indicated in the cases of two groups, the young drivers and the accident prone, both of whom have more than their share of accidents, the attempt to reduce the accident rate in the great undivided class must make intensive and continuous use of every weapon that can be thought of This majority has every conceivable kind of person in it and they have traffic accidents in every imaginable kind of way There is no panacea The problem cannot be solved by attack along any one line

I propose to discuss the problem in three parts Facts, What can we do about it? and Needed Information

FACTS

Individual Indifference

To me a most significant fact is the indifference we all display as individuals to this serious situation True, we talk about it, we hold meetings, we make speeches, and sometimes we even give money, but to get right down to it and apply everything we hear about bad drivers to ourselves is not done When we drive, all of us—drunk or sober—think we are good drivers and none of us expects to have an accident The reason for this is understandable· Of 28,270,000 motor vehicles about 43,470 [1] were involved in 37,800 fatalities in 1936 and proportionately there were probably about 1,400,000 vehicles involved in the 1,200,000 personal injuries in traffic accidents of 1936. The chance of my car or yours being in one of these accidents is slight If the average vehicle is driven 10,000 miles per year there are approximately 283 billion vehicle miles traveled or 202,143 for each vehicle involved in a personal injury accident This means, if you drive your car 10,000 miles per year, your chance, if you are an average driver, is one serious accident in 20 2 years. The plain mathematics of the situation is that no driver either consciously or subconsciously expects to have an accident now, but the possibility of only one personal injury accident in a lifetime is something to be concerned about

Accident Prone and Accident Free

It is well known that some individuals have the unhappy faculty of being on hand when accidents happen Recent investigations of the Bureau of Public Roads and the Highway Research Board in cooperation with the Connecticut Commissioner of Motor Vehicles demonstrates the presence in the driving population of a relatively small group of high accident individuals, a larger group of comparatively accident free individuals and the residue, which comprises the bulk of the drivers among whom the larger part of the accidents are distributed more or less according to chance The distribution is no doubt affected by ages of drivers, liquor, recklessness, physical conditions and in fact by all of the myriad factors that figure in accidents

A random sample of every tenth one of 408,000 drivers re-licensed to operate in Connecticut in 1932 resulted in a list of 29,531 drivers who were licensed through the six-year period 1931-36 and whose accident records were known In the six-year period these 29,531 drivers had 7,082 accidents that were reported However, 23,881 drivers had no accidents and 4,503 had one accident leaving only 1,147 who had two to seven accidents

According to Poisson's law of small

[1] Accident Facts 1937, National Safety Council

chances, if the 7,082 accidents had been distributed among the 29,531 drivers without regard to identities and personal histories, 23,234 of them would not have an accident, 5,572 would have had only one accident and 725 would have had two or more accidents There were, therefore, 647 more drivers who had no accident and 1,069 fewer drivers who had only one accident than would be expected if the accidents were distributed on this basis The better-than-to-be-expected results in these two groups indicate that there must be a fairly large group of drivers in the population who are more than ordinarily free from traffic accidents

And on the other hand, the fact that there were 422 more drivers who had 2 to 7 accidents than expected indicates the presence of a smaller group who are more than ordinarily susceptible to traffic accidents

The significance of these facts is not so much that there is an accident prone group, for that has long been recognized, but that this group is relatively small In the six-year period studied, only 1 5 per cent of the population appeared to be accident prone With longer experience this figure would presumably increase since no doubt some accident prone individuals might luckily escape with only one accident in such a short period as six years But even if we estimate that one quarter of the one accident group are also accident prone, we have accounted for only 5 per cent of the population and 30 per cent of the accidents

Of course, it is imperative that this group of accident breeders be reduced but even if they could be "liquidated" altogether, the larger part of the accident problem would still remain

It is true that these figures are based on study of a limited population in a single locality for a comparatively short period Other populations, accident definitions, and times would produce other figures However, I believe that this experience is sufficiently close to a cross section of the accident situation to justify the rather broad conclusions I have here set down

Identification of the Accident Prone

Comparison of the first three years of the six-year period with the last three years showed that, given the histories of a group of drivers in one half of an experience such as this, it is possible to predict the performance in the second half This demonstrates that if adequate driver histories are recorded they can be used to sort out the high accident individuals in a comparatively short period even though an average individual's chance of being in a personal injury accident is only once in 20 years

Far better than detecting accident prone drivers from their records after they have done much damage, would be to examine prospective drivers and determine in some way whether or not they might be expected to have this propensity To this end scientists have experimented with tests that require the operator to use skills that presumably are necessary or related to those that are necessary for good driving With the cooperation of the Iowa State College, the Harvard Bureau for Street Traffic Research and the Connecticut Department of Motor Vehicles the tests developed by Lauer at Iowa State College and DeSilva at Harvard were applied to about 3,000 drivers in Connecticut where their records could be ascertained The analysis of the data has not been completed but so far it appears that a subject's reaction to the whole of either set of tests will not indicate his propensity to accidents in general Whether or not individual tests may show leanings toward particular kinds of accidents has not yet been determined

Younger Drivers

Data from Connecticut, Massachusetts and the District of Columbia demonstrate that the younger drivers, ages 16 to 25 years, have nearly twice as many accidents involving death or personal injury as would be their share according to their number in the population In the Connecticut study of 2,467 fatal accidents it appeared that if the fatal accident rate among the drivers less than 25 years in age could have been reduced to the average rate of the whole population 291 fatalities or 11 8 per cent would have been avoided This fact points to a definite part of the population which needs and should be susceptible to educational methods Of course there is some overlapping of the accident prone and young driver groups

Multiple Causes of Accidents

No one can read even a part of the 1,715 case histories of fatal accidents collected by the Bureau of Public Roads and the Highway Research Board with the cooperation of the Yale Transportation Committee without being impressed by the innumerable combinations of conditions and circumstances relating to vehicle, road, driver, passenger, and bystander, which are involved in highway traffic mishaps It is seldom that a single cause can be assigned, in most cases at least three important factors contribute to the unfortunate result

Reports, Records, Laws, Rules, Signs

It would seem, without argument, that a primary requisite for traffic control would be accurate and complete reports and records of accidents, continuous accident histories of all licensed drivers, a reasonable, sensible and helpful system of laws and rules; and an adequate system of signs and signals for the information of the traveling public It would furthermore seem, in view of the total lack of significance of state lines to motor vehicle traffic that these various functions should be uniform throughout the United States In spite of the elementary nature of these postulates, and in spite of the fine work for a number of years of the Conference on Street and Highway Safety the report of the Secretary of Agriculture to the Congress exhibits a truly chaotic condition in these important safety requisites

Day and Night Accidents

If night driving could be made as safe as daytime driving, highway accidents would be reduced by one third and almost half of the lives lost might be saved As reported by Arnold Vey [2] in New Jersey in 1933, 44 per cent of the total accidents and 55 per cent of the fatalities occurred at night This is corroborated by the National Safety Council's "Accident Facts" for 1937 which reports that in 26 States 48 per cent of all personal injury and 60 per cent of all fatal accidents occur at night Approximately half of all accidents occur at night when the traffic is much less than in the daytime According to Vey, night traffic in New Jersey is about 20 per cent of the 24 hour traffic These facts mean, therefore, that daylight traffic must be four times as dense as that at night in order to make travel by day as hazardous as by night

In New Jersey in 1933 there were 1,185 fatal accidents, of which 651 occurred at night By elimination of the hazards due to darkness, 488 of the night fatalities or 41 per cent of the total deaths might have been avoided

In view of our groping efforts to lessen the traffic toll the conclusion that by overcoming the hazards of darkness the problem can be reduced by one third or

[2] "The Relation of Highway Lighting to Highway Accidents," Arnold Vey, Proceedings, Highway Research Board, Vol 14, Part I, p 429

more is startling Here is a point of attack that warrants almost any expenditure of time or money

There is evidence to indicate that for roads where the density of traffic justifies the cost, effective overhead lighting is being developed Data from New Jersey in Table 1 show that the lighting of one heavy traffic road did make day and night driving comparable

TABLE 1

Comparison of Day and Night Accidents on Lighted and Unlighted Roads in New Jersey

Route	Miles	Av No of vehicles per 24 hours	Day accidents per million vehicle miles	Night accidents per million vehicle miles
No 25, Lighted	4 22	32,000	3 10	2 61
No 26, Unlighted	6 52	8,800	2 42	7 70
No 26, Unlighted	2 27	8,400	2 08	8 80
No 26, Unlighted	3 90	8,400	2 79	7 55

Safeguarding the Highways

The type of highway that can be built in any given location depends fundamentally upon its earning power through charges against highway users and in some cases other beneficiaries In congested regions reasonable charges will provide four lane divided roadways, grade separations, limited access, lighting and all the other safety arrangements yet devised, but on thousands of other miles of main roads, raising the cost of such improvements would tax the highway user out of business As not even an approach to this condition is likely to be tolerated by public opinion, it remains that the only hope of improving the accident situation on huge mileages of roads and streets lies in persuading the drivers to operate carefully, and in improving those features that constitute hazards to reasonably careful drivers It is a fact that our so-called "inadequate highway system" somehow manages to carry an annual traffic of 300 billion vehicle miles with a personal injury accident rate of only 4 25 per million vehicle miles with fatalities at the rate of only one per 7½ million vehicle miles

WHAT CAN BE DONE ABOUT IT

Having thus assembled certain facts which are particularly pertinent to a broad picture of the condition that confronts us, what do they indicate can be done to improve the situation?

The United States, if not the whole world has enthusiastically adopted this form of speedy personal transportation, in which, I think, all will agree there are some inherent risks Our problem is to learn how to live with it in such a way as to minimize the hazards to life and limb[3]

It is apparent that there is no panacea for the hazards of highway transportation No single corrective measure can satisfy the need The majority of the accidents happen to the great body of the drivers in myriad ways and through almost infinite combinations of circumstances

It is evident that in so far as millions of drivers are concerned there is a large element of chance The long-time objective, therefore, must be the lowering of the accident rate level for those drivers that are not accident prone nor susceptible to accidents on account of lack of experience

[3] After T H MacDonald

But before discusing that problem, let us consider the cases of the youthful drivers and the accident prone

Youthful Drivers

There is no evidence as to why it is the younger drivers have so much more than their share of the accidents Whether it is because of lack of manual skill or lack of good judgment in the pinches we do not know It is probably some of both However we do know that in most States new drivers are required to demonstrate some degree of manual proficiency before being licensed and that very little attempt is made to make sure that they are taught those more extensive attributes of judgment and attitude that are necessary for good driving

However, one treatment is plain, better training for new drivers Just how this is to be accomplished is not to be gone into here It is enough to say that training and examination of new drivers must be undertaken on a hitherto unthought of scale The fine pioneer work in this field of the American Automobile Association must be vastly extended

Accident Prone Drivers

The case of the accident prone is not so plain First he must be isolated as an individual, then he must either be eliminated or reeducated so that his hazardous tendencies will disappear As has been shown it is possible from adequate records to select the group of drivers which contains the accident prone individuals How to select the individuals and what to do about them is a problem for continued psychological research Tests and measurements of physical and mental attributes that may affect a person's driving ability have been under intensive investigation, notably by Lauer and DeSilva for some time Although I do not think the applicability of these tests to pre-identification of hazardous drivers and to granting of driving licenses has been defined, there can be little doubt of their educational value, in that anyone should profit from knowledge of his own characteristics If one has defects in skills that are related to driving a car, knowledge thereof should at least tend to make him more careful

It is at least evident that an adequate system of accident reporting and recording is essential for control of the high accident drivers To do anything at all on this situation administrative authorities must have at their finger ends the histories of all drivers who have participated in accidents

Lowering the General Accident Level

The only apparent fact about the problem of reducing the accident rate for the great mass of unsegregated drivers is that to affect this situation intensive and continuous work on every possible angle must be prosecuted for a long time The brunt must be borne by the highway transportation industry (which takes in most of the population in one way or another) and government Neither can handle the situation alone and both will have to put some very large scale money and effort into it before the accident rate can be reduced to its lowest level

To discuss all of the possibilities would require a volume I shall only mention in passing what seem to me to be some of the fundamental factors in the situation

Drivers An entirely new attitude of mind toward the use of highways by fast moving vehicles is needed We still retain too highly individualistic a concept of our rights and duties Highway transportation at the speeds that are bound to prevail will always be hazardous and community safety as well as personal safety must become a first concern of the individual

It would help if we could all adopt the frame of mind that we must be in some way at fault ourselves whenever we get into a risky situation or are involved in a mishap There is considerable truth in the thought that "Good drivers do not get into tight places" There are some people—I do not know how many, I have only met a few—who have a judicial attitude toward their own acts and can adopt that point of view, but there are plenty to whom it is mentally impossible to take that position Nevertheless continual bearing down upon such points should ultimately have some effect

Better driving habits must in some way be inculcated It is obvious that we cannot be frightened or intimidated into better habits The apathy of the individual, previously mentioned as being accountable for by the low accident expectancy of the average driver, demonstrates this The only answer I can think of is, better training of the new drivers and reeducation of the old ones Better training of new drivers will in the course of time account for the whole population and is therefore of tremendous importance In the case of the present older drivers some system of tests that will show them their weaknesses in places that may affect their driving ability may prove to be effective Of course it would be a huge task to test 28 million drivers, but if such ideas are found to be effective we must cease talking about costs being prohibitive

Pedestrians Two out of five traffic accident deaths are pedestrians[4] Certainly attack on this phase of the problem is clearly indicated The record[4] shows that 52 per cent of the pedestrian accidents occur between 6 P M and midnight, which indicates that light must be an important factor in their occurrence A significant fact reported recently by Michael A Conner, Connecticut Commissioner of Motor Vehicles is that, of 1,238 pedestrians killed in Connecticut during 1932-1936 (inclusive) only 48 were operators of motor vehicles This shows the necessity of impressing upon the non-driving public the difficulties of handling motor cars in heavy traffic and the fact that although a walker may clearly see an approaching vehicle, conditions may be such that the driver may not see him It is quite possible that a man may be within the visibility range of a car's headlights and still the driver may actually not see him until it is too late All drivers are aware of this possibility

The fine work being done with school children should be supported and extended. It will not only save the lives of juveniles now but should lower the accident rate when those who are children now are grown up

Vehicles· It is perhaps needless to say that the driver is entitled to a vehicle which is originally safe for reasonable use, and that thereafter he has a responsibility to others as well as to himself to keep it in safe condition The producers of motor cars are well aware of the necessity of embodying in their design every practicable feature likely to contribute to safe operation, and improvements in this respect are made continually I shall only mention three items that seem to me to offer possibilities (1) Better range of vision for the driver, (2) Obviation of the time lost in moving the foot from the accelerator to the brake pedal in emergencies, and (3) Elimination of headlight glare along with increase in visibility at night On thousands of miles of road the vehicles must continue to carry their own illumination and the need for something better than we have is obvious Although many headlighting improvements have been made, it is still dangerous to meet another car at night

[4] Accident Facts 1937, National Safety Council

when all one can see is the lighted space between the two cars and that made indistinct by the glare of the approaching headlights

Roads The normally careful driver is entitled to roads safe for reasonable use He is entitled to more than that, he is entitled to a factor of safety that will give him some measure of protection against the hazards over which he has no control Of these I know of three [5] (1) The acts of other drivers, (2) The acts of pedestrians (particularly when one steps into your path from behind a parked car), and (3) blind intersections

Much has been learned about elimination of hazardous features of highways, width of lane, sight distance, super-elevations, shoulder width and treatment, ditch sections, skid resistant surfaces, and gradient are details of design that are now studied much more carefully in relation to safety than in the early days of the motor vehicle If what we now know is desirable could be applied at once to all old as well as new roads highway accidents could be greatly reduced

Since the greater part of the rural roads cannot be made accident-proof, or even fool-proof, the only road solution to the accident problem is to build into the type of road justifiable on economic grounds at a given location, all of the known safety features applicable to its type or class This will involve due attention to the prevailing and to be expected customs as respects speed and vehicle characteristics On existing improved roads it will involve careful study and treatment of specific locations potentially and actually hazardous from the standpoint of modern operating practice Dependent on traffic conditions improvements for safety should range from treatment of hazardous spots to the construction of limited access super-highways

[5] After H C Dickinson

Traffic Control Much can be done to lower the accident rate through proper control of the traffic, but there is an urgent need for unification of practice throughout the nation The driver is certainly entitled to uniform rules, regulations and practices in everything that affects driving habits he is entitled to freedom from conflicting and meaningless laws and regulations Certainly when this objective is reached it will be reasonable to expect improved conditions

Law enforcement is a necessary adjunct to traffic control but it is axiomatic that no law affecting the entire population can be enforced without popular support It follows therefore that regulatory measures that are needed for safety must be sold to the public In this connection it must be recognized that most of the acts in violation of motor vehicle laws or ordinances are not criminal in nature, and that treatment of such offenders from the criminal standpoint will not promote popular support for law enforcement

Remarkable results have been secured in Evanston and several other cities by using the police power from the standpoint of accident prevention, and it is through this type of activity that the law enforcement agencies may contribute most toward lowering the accident rate

More patrolmen on rural highways are urgently needed Drivers en masse cannot be influenced much by reading the record of arrests, convictions, and license revocations in the daily papers, but most of us can be impressed into carefulness by frequent sight of uniformed policemen

RESEARCH

In discussing corrective measures little was said about the details of how they might be accomplished This was partly because in many respects little or nothing can be said until more knowledge is available For instance it was said that better driving practice should be inculcated, but who knows with certainty just what constitutes the best driving, or who knows what should be done to solve the problem of vehicle headlighting

It would be easy to go ahead and generalize on the subject of needed research and compile a long list of matters that should be studied However, what is needed at this time are concrete suggestions for research projects that can be undertaken now with hope of constructive results

From the research point of view the driver is the most interesting factor as he is the element about which we know the least and which is the least under control, but there are several aspects of the road and vehicle that should be mentioned

Road Factors

Design In drawing plans for a given road the designer must take into account as best he may the volume of traffic and the expected speeds Better understanding is needed of the inter-relations of speed, traffic volume and day and night visibility, and such roadway characteristics as gradient, curvature, surface friction, sight distance, width, and cross section Some suggested research projects are

(1) Observations of vehicle behavior on curves of different radii and superelevation at different speeds

(2) Study of road conditions and accident records in every highway jurisdiction to locate hazardous spots

(3) Observations of conditions and distances by which objects variously placed can be seen or not seen at night by drivers of moving vehicles

(4) Experimental lighting of a sufficient number of roads and hazardous spots of known accident history to establish criteria for the use of overhead lighting

Surface Slipperiness The resistance of pavements to skidding is of great importance Recently apparatus has been devised by which the frictional resistance between surfaces and tires can be measured, but only comparatively few surfaces have been tested Every road building authority could improve its practice by measuring the frictional properties of the surfaces it has built and is now building, and by constructing experimental sections for the purpose of determining upon the best design

Guard Rail Tests Numerous uncorrelated tests have been made of different types of guard rails There is need for a correlation study of all of them with a view to devising a standard method of test by means of which guard rail designs can be evaluated

Warning and Informational Signs There is a great lack of uniformity in the methods used to convey important information to the speeding driver The standards of the Association of State Highway Officials do not go far enough, stripes and other markings on the pavement should be included as well as roadside signs There are great differences of opinion over the use of stop signs, and over the locations and spacing of all signs A national survey is needed of the use and observations of all kinds of signs with a view to producing a complete standard code

Vehicle Factors

Headlighting The need for portable lighting equipment that will eliminate glare and give greater visibility has already been stressed This is a form of research for which the reward of success will be very definitely in lives saved

Fleet Accident Records The preliminary survey of fleet accident records made by the Bureau of Public Roads and the Highway Research Board in cooperation with the Society of Automotive

Engineers and the Harvard Bureau for Street Traffic Research, indicates that valuable information concerning the relation of vehicle characteristics to safety could be secured from detailed study of such records

Inspection of Vehicles It is generally agreed that lack of maintenance of certain mechanical details may create hazards However, the extent to which vehicular details have contributed to accidents is not known, nor has the value of the compulsory inspection of vehicles carried on in many jurisdictions been demonstrated A thorough study of all information bearing upon this factor is recommended

Operators

Here we come to the factor that is complicated by all the idiosyncrasies of the human race It is a field of research that will tax the ingenuity of the psychologist, the physicist, the engineer, and many other scientists

Before better driving habits can be inculcated we must know what constitutes good driving practice, and the obvious place to start to learn that, is to study drivers' behavior as it is to be found now.

Drivers' Behavior. Without attempting to exhaust the possibilities the following studies are suggested

(1) Study of a large number of drivers' histories, both good and bad in order to ascertain the attitude of mind displayed toward the job of handling a motor car in traffic—suggested by Dr Ralph Lee

(2) Study means of identifying reckless and hazard creating drivers. Dr. H C Dickinson has suggested that if in a given area a large corps of observers note the license numbers of all cars they see doing one or more of a short list of dangerous acts, it should be possible to sort out many repeaters who habitually cause hazardous situations A brief preliminary study has indicated the validity of this assumption. This method should be checked extensively enough to demonstrate whether it is a feasible method of identifying bad drivers before they have accumulated a costly accident record If the method should prove to be sound the next step would be a study of what to do about them

(3) Studies of motorists' habits with respect to speed under specific conditions, reactions to signs and signals, and reactions to various physical conditions such as steep hills, sharp turns, narrow roads

(4) It has been suggested by Dr H C Dickinson that valuable insight could be secured into what constitutes good and bad driving by equipping cars with automatic speed recording devices and running extensive tests with drivers of known records, both good and bad This idea was extended by Dr Ralph Lee who suggests that the automatic recording device be arranged to give a record of the movements of the steering wheel, brake, clutch and accelerator

Driver Training· Considering its infancy much good work has been done on this educational problem Well developed methods of demonstrated efficacy are available[6] However, it is hardly to be expected that the last word has been said and if development of educational methods in this field follows the usual course, it will only reach perfection over a term of years through the research and experimentation of many individuals The most pressing need at this time is for the application of what we know today

Driver Tests The early detection of accident prone and hazardous drivers is so important that investigators should be encouraged to go ahead and exhaust the possibilities of physical and psychological testing

Alcohol and Motor Vehicles· Although all jurisdictions have drastic penalties for driving while under the influence of

[6] American Automobile Association

liquor and in spite of the fact[7] that accidents traceable to this influence have been increasing, the penalties are not often applied

This is not necessarily due to laxness of enforcement, but rather to the difficulty of proving in court that the person was intoxicated at the time the accident occurred Unfortunately, the moral certainty of the arresting officer is not sufficient proof in the eye of the law For this reason the charge is often changed so that the offender can be convicted of some lesser offense rather than be permitted to go scot free

Although it is practically impossible to prove intoxication from symptoms alone, in the lesser degrees, the debatable cases make driving dangerous for the sober drivers Even before a person is intoxicated, in the common interpretation of the term, there are effects that decrease driving ability—slower reactions, lesser coordination, narrowing of attentional field and that increased self-assurance which makes one think he is performing brilliantly while in fact doing anything but that

A definite method of determining the degree to which a person's actions are influenced by a certain amount of alcohol is needed Three methods of test are receiving research attention Analysis of the blood, the urine, and the breath The blood test is the most positive but it requires the voluntary cooperation of the subject The other two need further correlation with the blood test And for all three the percentage of alcohol found present must be more definitely related to traffic hazards Also the problem of making use of such scientific methods in judicial procedure needs thorough study

Research is needed to determine the real part played by alcohol in traffic accidents "During 1936 the reported percentages in 26 States of 'had been drinking' or 'intoxicated' participants in fatal accidents averaged 7 per cent for drivers and 11 per cent for pedestrians Cities reported an average of 8 per cent for fatal accident drivers, and 10 per cent for pedestrians killed That these percentages understate by an unknown amount the true situation is the belief of most traffic authorities How high is the true percentage? That is the question "[7]

Spot Investigations

Cutting across all of the divisions of the accident problem—roads, vehicles, drivers, pedestrians—is the need for study of accidents themselves After all it is the combinations of circumstance which cause accidents that we must learn about in order to devise remedial measures

Much has been learned from the recent study of public records of fatal accidents, but it is felt that much more can be learned from study of the reports of technical investigations of accidents made on the spot immediately after the occurrence A competent technical observer should note many details, the significance of which would escape the police officer

Since the combinations of factors causing accidents seem to be countless, adequate prosecution of such a research project will require examination of a large number of cases

CONCLUSION

In conclusion let me merely repeat that the big job will be to lower the accident rate among the great body of drivers, most of whom are reasonably careful, and whose serious accident expectancy is slight but whose numbers are so great that in the aggregate they produce a heavy toll in serious injury and loss of life

This is no small problem

[7] Accident Facts 1937, National Safety Council

DISCUSSION—THE HIGHWAY SAFETY PROBLEM

MR MAURICE HOLLAND, *Director, Division of Engineering and Industrial Research, National Research Council* Mr Chairman, I have knowledge of this specific subject before you I am the voice of the pedestrian I live in New York City I have never driven a car I don't own one I have driven airplanes during the war and later I have definitely enlisted the sympathies of a group of pedestrians to form a Pedestrians' Protective League and although some of you plutocrats may be automobile owners or drivers, you are also pedestrians I do not know how many of you have tried to get across the streets of New York City when a taxi comes tearing round a corner with blast of horn or screeching brake Your life is not worth a nickel I gather that if we could mobilize in New York a considerable group, we could get a bunch of lawyers to defend our cases and to also educate our group in how much of that road is open to us, the pedestrians, as is the traffic for the motorist—that in a good many instances we would have New York's "Finest"[1] on our side

I had an experience recently at Sixth Avenue and Thirty-Ninth Street In trying to get across the street with the green light—I never cross except with a light—half a dozen impatient automobile drivers had to beat the gun before the traffic got across, but there was an alert cop there who put those boys where they belonged Everybody got across the street safely It was my particular pleasure to cite that man to the Commissioner of Police of New York City and shortly after he was promoted for his vigilance in the interest of pedestrians I do not know how the ratio is of pedestrians, automobile owners and drivers in New York City, but I have only to walk some 30 blocks in New York City twice a day and I am here to put on record the voice of the pedestrian who is going to rise in his wrath and use the same protective measures that you have on automobiles—safety devices, or organizations to see that he gets a square break

[1] New York City Police

MR C M JOHNSTON, *Bureau of Public Roads* Pedestrians, particularly in the South with its colored people, should wear lights or some sort of reflectors when walking on the highways at night It is not always the motorist's fault when he fails to see a pedestrian in the dark The pedestrian takes the attitude that the motorist can always see him It is not true, not even when the automobile has bright lights

REPORT OF DEPARTMENT OF SOILS INVESTIGATIONS

C A HOGENTOGLER, *Chairman*

Progress has been made during the past year in the following lines of endeavor (1) Stabilized roads, including both base and surface courses, (2) Fill construction, and (3) Tests of soils for foundation purposes

Specifications have been prepared and submitted to the American Association of State Highway Officials The following have been approved by the Committee on Materials of the Association

(1) Standard Specifications for Materials for Stabilized Surface Course, including

Type A—Sand—clay mortar

Type B—Coarse graded aggregate

Type C—Gravel, stone or slag screenings or sand

(2) Standard Specifications for Materials for Stabilized Base Course, including

Type A—Sand—clay mortar

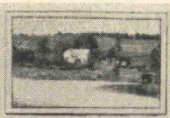
Type B—Coarse graded aggregate

Type C—Gravel, stone or slag screenings or sand

(3) Standard Specifications for Material for Use in Embankment Construction

The essential features of these developments are given in the following seven plates which are taken from the exhibit prepared by the U S Bureau of Public Roads for the convention and Road Show of the American Road Builders' Association in Cleveland, Ohio, in January 1938

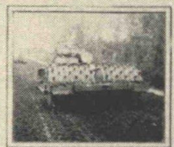
Four reports presented at the open meeting of the Department on Tuesday, November 30, are appended to this general report



Type No. 1.- GRADED MIX, FOR LIGHT TRAFFIC, BUILT WITH BEST LOCAL MATERIALS AVAILABLE. The main purpose of the type is to provide an inexpensive all-weather surface for the light traffic on the land-service roads. This surface may become dusty or muddy and portions may get rough or remain smooth. It may be built to serve all-year-round travel at costs ranging up to $1,500 a mile.

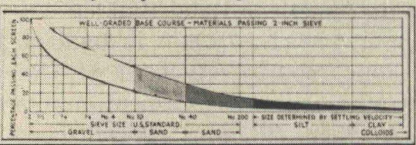
Type No. 2.- GRADED MIX, FOR MEDIUM TRAFFIC, BUILT WITH A DESIGNED AND PROPORTIONED MIX. This type provides adequate service for heavier traffic than Type No. 1 because the surface is wider and thicker, the mixture is designed carefully, and the various steps in the construction are controlled so as to produce a dense mix. This surface may be built for costs varying from $1,500 to $2,500 a mile

Type No. 3.- GRADED-MIX BASE WITH A BITUMINOUS SURFACE TREATMENT FOR HEAVIER TRAFFIC THAN TYPE No. 2. This is a compact, smooth-riding surface, with considerable resistance to skidding, which is free from mud or dust at all seasons of the year. This substantial surface treatment on a properly-constructed base may be built for $3,500 to $5,000 a mile. When all new material is used the costs may exceed $5,000

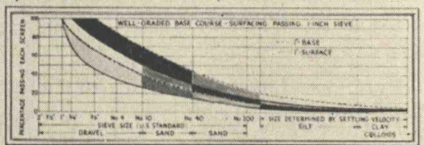


Type No. 4.- STABILIZED SOIL BASE WITH A BITUMINOUS-MAT SURFACE TREATMENT - FOR HEAVIER TRAFFIC THAN TYPE No. 2. The poorly graded soil base is stabilized with bituminous binder or Portland cement. This surface when properly built is suitable for relatively heavy traffic. The meager data available indicates that it may be built for prices varying from $4,500 to $7,000 a mile. If new aggregate is required to improve the grading of the fine-grained material, the cost may exceed $7,000 a mile

TYPES NUMBER 1, 2, AND 3 require WELL-GRADED MATERIALS such as are indicated on the chart below, within the typical grading band, for road surfaces with clay binders enclosed within the RED dashed lines. The grading band for good base materials with clay binders is enclosed by the BLUE full lines



PARTICLE-SIZE ACCUMULATION CURVE



PARTICLE-SIZE ACCUMULATION CURVE

BEWARE! GOOD SURFACES MAY BECOME UNSTABLE BASE COURSES

BASE COURSES.- All good surfaces do NOT make good bases because the plasticity index and liquid limit are generally LOWER for a base than for a surface: Except for those gradings shown within the overlapping bands on the chart above, good base soils are coarse and contain smaller amounts of silt and clay. Highly stabilized surfaces often become unstable when covered with impervious bituminous surface mats and used thus as bases

THEREFORE, THE DESIGN OF A BASE COURSE IS DIFFERENT FROM A SURFACE

SURFACES.- In the construction of surfaces, especially Type No. 1, more latitude may be used in the selection of materials than in the base courses for high-type surfaces. The reason for this is that an error in the design may be corrected later by proper maintenance measures. In contrast, an error in the design of a base course means failure of the pavement laid thereon. A road surface, exposed to evaporation, requires more clay to hold sufficient film moisture to insure adhesion of the particles. On the contrary, evaporation from a base course is cut off by the impervious surface. THEREFORE, THE INCREASED MOISTURE in a GOOD SURFACE used as a base course causes all but the best of clays to swell and BECOME UNSTABLE

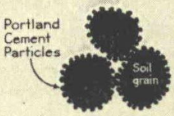
Figure 1. Four Types of Low Cost Surfaces which May Be Stabilized by Proportioning the Materials or by Treating with Admixtures

The poorly-graded bases under the Type No. 4 surfacing, described on the previous panel, may be stabilized by asphaltic, tar, and Portland-cement binders

THESE BINDERS SERVE A DOUBLE PURPOSE:

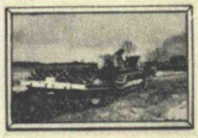
1.- They prevent the clay fraction from absorbing water and thereby softening the soil mass
2.- They bind the soil particles together
Thus they produce a **DENSE** and **STABLE, WATERPROOFED SLAB,** but **NOT** a rigid slab

COMPACTION TESTS, described in a previous panel, are used to determine: (1) the proportions of the impregnating materials and (2) the densities to which the slabs are to be compacted

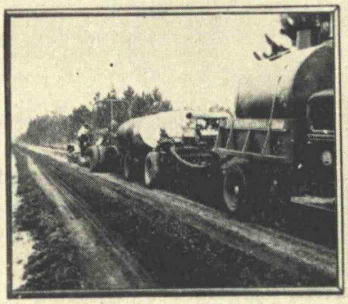


PROPER MIXING is possible when the materials are dry and powderlike so that when brought into close contact by mechanical manipulation, the Portland-cement particles thoroughly coat the soil grains. This may be accomplished by road or plant mixing

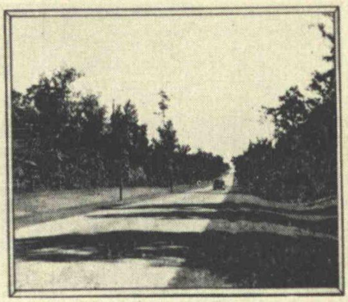
NEXT the predetermined **OPTIMUM AMOUNT OF WATER** is added and the mixture is made dense by **COMPACTION** with the aid of sheepsfoot multiple wheel and similar types of rollers



BITUMINOUS MATERIAL COATS THE SOIL PARTICLES MORE READILY after the air films have been removed **BY WETTING THE SOIL GRAINS**
THEREFORE, WATER IN VARIOUS AMOUNTS IS USED TO AID IN THE PROPER DISTRIBUTION OF TARS, ASPHALTS, AND EMULSIONS THROUGHOUT SOIL MIXTURES



Road-mix machine building an emulsion gravel base on U.S. Route 1, in Georgia



Bituminous surface treatment on U.S. Route 90, in Florida

Figure 2. Stabilization of Poorly Graded or Fine Graded Soils

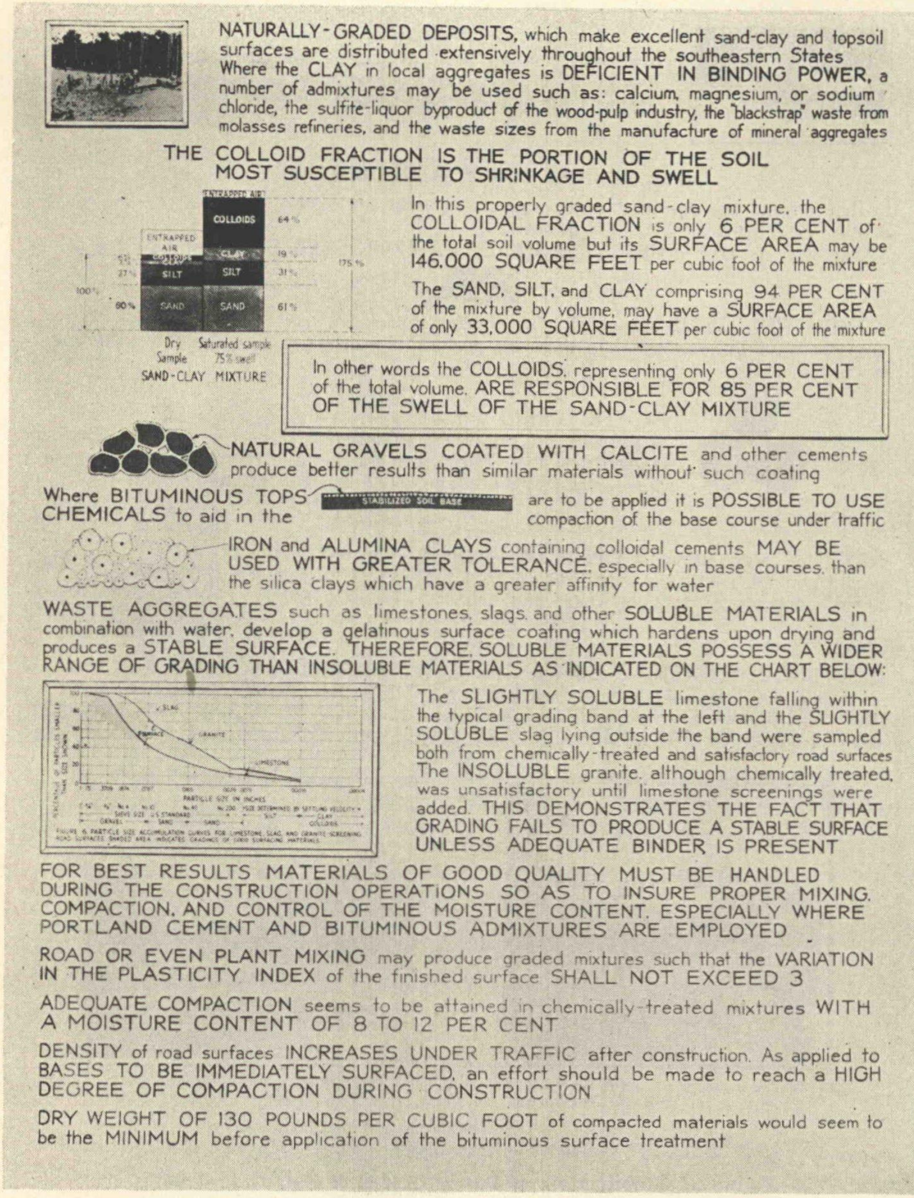
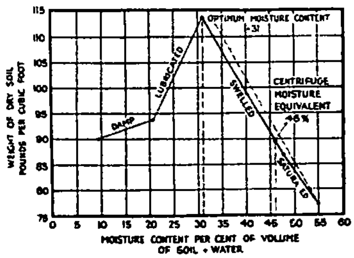
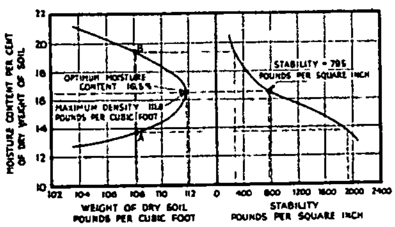


Figure 3. For Stability and Long Life Use the Best Materials Available within the Allowable Cost Per Mile of the Road



Consider the typical soil sample shown in the diagram at the left
Up to 20 7 per cent moisture - The soil is damp and the moisture films are highly-cohesive but have sufficient friction to prevent high density From 20 7 to 31 1 per cent moisture - The soil reaches maximum density
The moisture films have less cementing value but greater lubricating properties From 31.1 to 47 7 per cent moisture - Swell occurs, but stability and density drop The sample is wet
Above 47 7 per cent moisture - Water gradually replaces the air content until at 54 3 per cent moisture, the soil becomes completely saturated

RELATION BETWEEN DENSITY MOISTURE CONTENT, AND STABILITY



In the typical diagram at the left stability drops off rapidly as moisture contents are increased above the optimum and vice versa
At every density less than the maximum there is a moisture content below the optimum at A corresponding to one above the optimum at B, which the wet soil can attain without changing volume
BUT THE CORRESPONDING STABILITY AT A IS 1900 AND AT B IS 270 POUNDS PER SQUARE INCH

SUITABILITY OF SOILS FOR FILL CONSTRUCTION DEPENDS UPON THE DENSITY TO WHICH THEY MAY BE COMPACTED

HEIGHT OF FILLS							
10 FEET OR LESS *				MORE THAN 10 FEET			
Maximum Dry Weight	Approximate Bureau Public Roads Classification	Rating	Maximum Field Compaction Requirements	Maximum Dry Weight	Approximate Bureau Public Roads Classification	Rating	Maximum Field Compaction Requirements
Pounds per cubic foot			Per cent of Dry Weight	Pounds per cubic foot			
89 9 and less	A-5 A-8	Unsatisfactory	----				
90.0 - 99.9	A-5 A-8	Very poor	95	99 9 and less	A-5 A-8	Unsatisfactory	----
100.0 - 109.9	A-6 A-7	Poor	95	100 0 - 109.9	A-6 A-7	Very poor	100 0
110 0 - 119.9	A-4	Fair	90	110 0 - 119.9	A-4	Poor	95.0
120 0 - 129.9	A-3 A-2	Good	90	120.0 - 129 9	A-3 A-2	Fair	90 0
130 0 and more	A-1	Excellent	90	130.0 and more	A-1	Good	90.0

* For fills 10 feet or less in height the soils should have liquid limits (L L) not greater than 65 and plasticity indexes not less than 0 6 L L minus 9 0

Figure 4 Density of a Compacted Soil Depends upon the Thickness of the Moisture Film Surrounding the Soil Particles

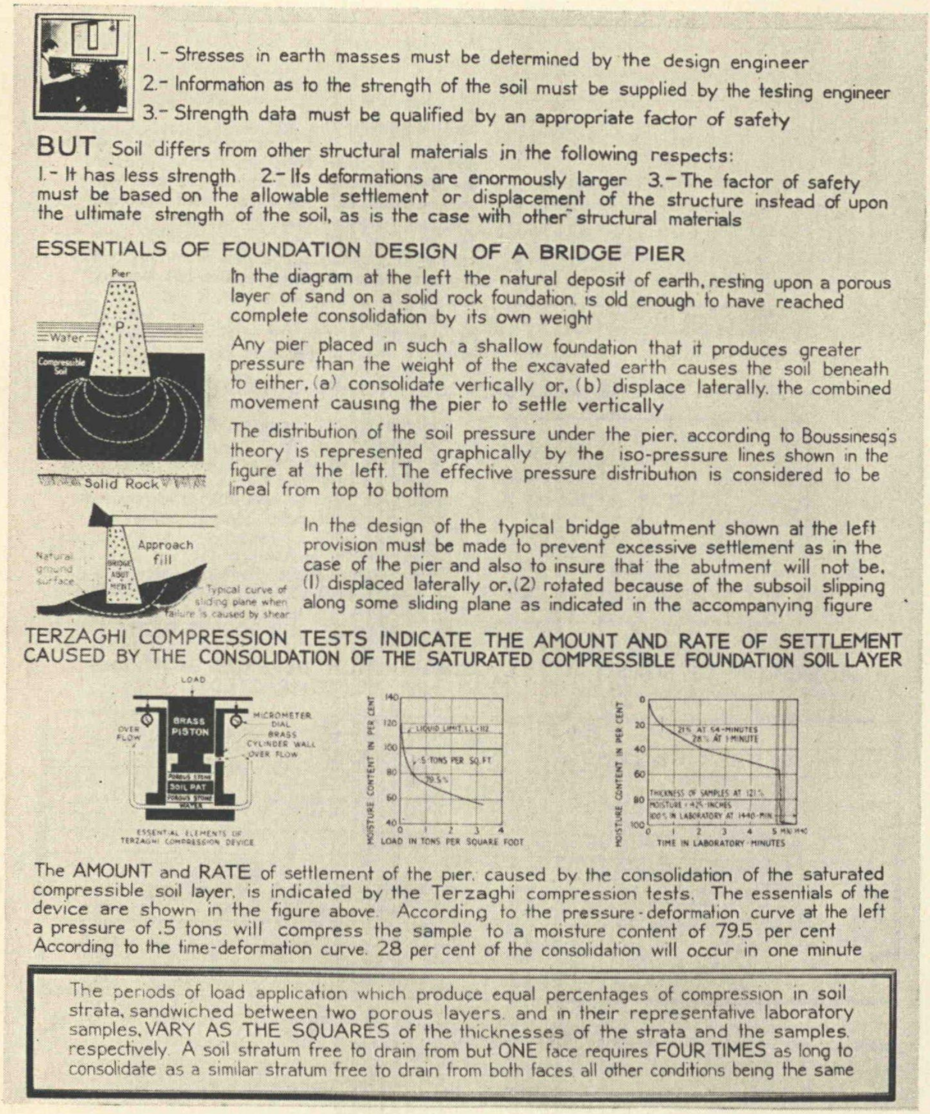


Figure 5. Soil Test Data Yield Like Information when Analyzed in Accordance with the Same Engineering Principle Used for Other Structural Materials Such as Concrete, Wood and Steel.

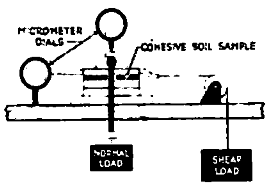
The shear strength of the soil depends upon the cohesion c, the angle of internal friction ϕ, and the pressure p upon the plane of shear, in accordance with the basic equation:

$$s = p \tan \phi + c \text{ ——————————————————— (1)}$$

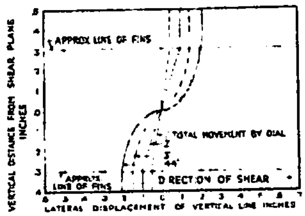
The soil must be deformed before shear strength is developed. In some soils, the horizontal deformations required to develop the ultimate shear strength indicated by test, causes the sample to be deformed as much as 60 per cent of its thickness. The characters of these deformations are shown below in the upper right hand figure

THEREFORE, the ultimate $c + \phi$ determined by test cannot be used but instead values at which ALLOWABLE deformatioms occur, and consequently relations of deformation to $c + \phi$ are necessary

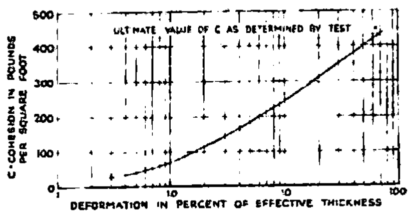
The essential features of a simple device used for determining the shear strengths of cohesive soils is shown in the diagram below

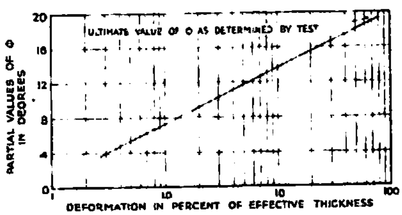


The chart below shows the data obtained for the maximum horizontal deformations of 0.1, 0.2, 0.3 and 0.44 inches in tests of samples 1-inch thick



FOR AN ACCURATE CONCEPTION OF THE EFFECT OF THE STRESS-STRAIN RELATION THE SAMPLE THICKNESS MAY BE TAKEN INTO CONSIDERATION BY EXPRESSING THE HORIZONTAL DEFORMATIONS AS PERCENTAGES OF THE VERTICAL THICKNESSES OF THE SAMPLES BETWEEN THE EDGES OF THE FINS OR GRIDS





The curves in the two figures above represent test values on a sample of clay soil 0.2 of an inch thick between fins; moisture content of 27 per cent; liquid limit of 43; and a plasticity index of 24

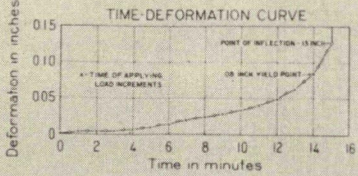
IT IS EVIDENT THAT THE POSSIBLE LATERAL DISPLACEMENT OF A BRIDGE ABUTMENT ON A SOIL LAYER 6 INCHES THICK MIGHT NOT BE DANGEROUS WHEREAS THE LATERAL DISPLACEMENT ON A SOIL LAYER 6 FEET THICK MIGHT BE DISASTROUS

Figure 6. Shear Strength Determines the Resistance of Soil to Lateral Displacement from Beneath Piers and Along Planes Beneath Abutments

The SHEAR STRENGTH of the soil depends upon the COHESION and the ANGLE OF INTERNAL FRICTION of the soil according to the basic equation:

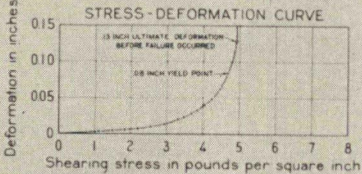
$$s = p \tan \Phi + c \qquad (1)$$

in which s = shear strength or stress in pounds per square inch
p = pressure normal to shearing plane in pounds per square inch
Φ = angle of internal friction in degrees
c = cohesion in pounds per square inch

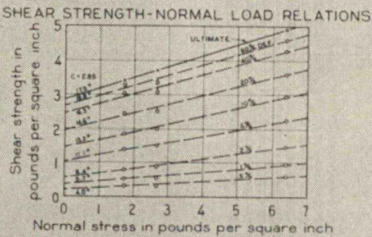


The time-deformation and stress-deformation curves at the left show the results of shear tests on a clay soil under a normal pressure of 6.8 pounds per square inch and at a single moisture content.

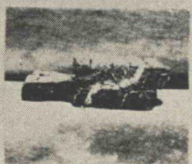
The time-deformation curve shows the relation of the horizontal deformation of the sample to the periods of shear (horizontal) load applications. The ULTIMATE SHEAR STRENGTH is computed by dividing the next to the last horizontal load by the area of the unsheared portion of the sample at the time of application of the last load.



The relation of shear stresses to horizontal deformations for any constant load may then be shown by the STRESS-DEFORMATION curve at the left. This shows how the shear stresses increase with the deformation until the maximum shear strength that would NOT cause failure was reached at a deformation of 0.08 inch.



By testing samples under different vertical loads, plotted as normal pressures, the relation of shear strength to normal pressure may be obtained as shown by the TOP line in the diagram at the left. The relations for deformations less than the ultimate are shown by the lines BELOW.



The photograph above shows how a vertical plane, in a sample before a test, was distorted after the sample was sheared to failure.

Design data for all other engineering materials except soils include stress-strain relations showing the ultimate strengths and deformations at which failures occur as well as the relation of stress to deformation for all points below the ultimate. In the case of soil materials it has been customary to use principally the value for the ultimate shear strength together with the corresponding angle of internal friction and the cohesion. As a result entirely erroneous conclusions have been reached with regard to the shear-strength relations where the deformations were less than the ultimate.

Available data indicate that the horizontal deformations of a soil under test are caused largely by accumulations of distortions distributed vertically through the sample. CONSEQUENTLY, THE MAGNITUDES OF THE RECORDED DEFORMATIONS ARE DEPENDENT MAINLY ON THE THICKNESS OF THE SAMPLE.

Figure 7. Shear Tests Show the Ulimate Strengths and Deformations at which Failures Occur as Well as the Stress-Deformation Relations Less than the Ultimate

REPORT OF COMMITTEE ON METHODS OF EXPLORING, SURVEYING AND SAMPLING SOILS FOR HIGHWAY PURPOSES

By Frederick J Converse, *Chairman,*
Assistant Professor of Civil Engineering, California Institute of Technology

SYNOPSIS

Methods of obtaining undisturbed soil samples are described in detail for eight States California, Iowa, Kansas, Louisiana, Michigan, New York, Ohio and Wisconsin Allowing for altering circumstances in various parts of the country, there is still widely differing opinion as to what constitutes satisfactory sampling The two principles of sampling always applicable are that the explored prism should be adequately covered, and that samples should be as nearly as possible undisturbed Because there is no truly satisfactory equipment yet available, it is hoped that highway departments will include a study of the problem in their programs of research

The methods of exploring, surveying and sampling soil for highway purposes may be divided into two general classes The first includes the determination of the physical characteristics of the soil as related to its performance in a reworked or compacted condition The second division includes principally the determination of the bearing capacity and permeability of the undisturbed foundation material

The first division has received a great deal of attention from research workers in the highway departments, and methods of surveying and sampling have been developed to a very satisfactory state The need for extensive work in the second division has not yet been so important in most States, but the development of high speed highways, with their deep cuts, high fills and larger bridges, is making the question of stability of foundations one of ever increasing importance to the highway engineer

Stability determinations are based on tests of undisturbed soil, and consist mainly of the determination of shear and compression strengths, density, and permeability These are laboratory tests, and are usually not performed on the material in place It is necessary, therefore, either to obtain undisturbed samples for use in the tests, or to devise new methods of getting equivalent values for the material in place Although the latter procedure has great possibilities of ultimate success, the former is farther developed at this time, and is giving results which are at least approximately accurate

The work of the committee this year has been directed toward extension of the general knowledge in this latter field By means of a questionnaire, a survey of the existing equipment now in use by the highway departments of the various States was made, particular emphasis being placed on critical analysis of equipment for determining undisturbed samples

The generous response from the State highway departments indicates much interest in the subject, but reveals that few have found it possible to extend personnel and equipment to include such investigations Of those reporting, California, Iowa, Kansas, Louisiana, Michigan, New Mexico, Ohio, and Wisconsin attempt to obtain undisturbed samples The methods used by these States will be described in detail

TEST PITS

For shallow depths the practice of digging pits and carving out a sample is followed in several States The practice in Ohio, as reported by Mr Litehiser, is as follows

"In obtaining undisturbed samples, a test pit of dimensions approximately 6 ft by 6 ft is excavated and samples taken at the desired depths When a sample is to be taken, a mound of undisturbed soil is left in the center of the pit and the remaining portion excavated to a depth of 11 in below the top of the mound This mound is then carefully trimmed until a cylinder of undisturbed soil 11 in high and 5½ in in diameter is left standing in the bottom of the pit A cardboard cylinder mold 12 in high and 6 in in diameter is placed over the cylinder of soil, and the space between the soil and the cardboard filled with hot paraffin When the paraffin has cooled, the cylinder of soil is cut loose from the base, inverted, and the other end sealed with paraffin

"When the test pit reaches a depth of greater than 6 ft, cribbing is used It is sometimes necessary to use pumps to keep the water out of the test pit

"Undisturbed samples are stored and carved for testing in the moist room at the laboratory

"Taking samples by this method insures getting as true undisturbed samples as possible Disadvantages are that this method of obtaining undisturbed samples is quite slow and expensive

"It is impractical to obtain samples from depths greater than 20 or 25 ft by this method"

Mr Kushing of the Michigan State Highway Department reports that·

"Where undisturbed samples are required, it has been the Department's practice to obtain these from a test pit rather than from a boring In the case of clays a large sample (¼ to 1 cu ft) is carved from the body of soil, trimmed, and coated with paraffin The sample is packed carefully to avoid breaking the paraffin seal and taken to the laboratory The sample is weighed in the field before and after coating with paraffin and these weights checked when the laboratory tests are run

"In the case of granular materials undisturbed samples have been successfully obtained by carefully forcing a seamless tin sample can into the sand or gravel until it is completely filled The can is raised on a square pointed shovel or trowel, inverted, and the top struck off with a straightedge The sample is weighed and sealed and taken to the laboratory for testing This method of obtaining undisturbed samples of granular materials has been used successfully even in gravel deposits, but is particularly suited for sand

"The methods used for obtaining undisturbed samples require the digging of a test pit for deep foundations, but in the case of subgrade investigation where samples can be taken at or near the surface, the expense is very small It is then possible to take a much larger number of samples and obtain a more comprehensive picture of the soil conditions"

It is recognized by all that this method of sampling, while satisfactory for occasional samples near the surface, is slow and expensive where many samples are required and deep pits are necessary Pipe samplers of many forms have been used extensively for deep sampling The present stage of the development and practical use of such devices is disclosed by the results of the Committee's survey

DRIVEN TUBE SAMPLERS

Mr Roettiger reports that in Wisconsin:

"Attempts have been made to obtain undisturbed samples by driving sharpened sections of pipe with the idea of cutting a core from the soil This has not been satisfactory because it compresses the soil at the circumference of the sample It also squeezes out some of the moisture and the sample is not representative of the natural condition of the soil"

"The regular practice of the Michigan State Highway Department has been described in some detail in a paper entitled "A Penetration Method of Measuring Soil Resistance" in the Proceedings of the A.S T M for 1935, Part II Samples are taken in a 1¼-in steel core barrel driven by a falling weight The core barrel has a removable cutting edge of hardened steel The primary purpose of obtaining samples in this way is to measure the resistance to penetration However, the samples obtained are sealed in the core barrel with paraffin and taken to the laboratory for testing The core is punched out of the core barrel and a transverse shear test, also described in the above article, is run The comparison of the laboratory shear test and the field penetration indicates that for plastic soils the sample obtained is relatively undisturbed In the case of more porous soils, it is believed that the core is compressed somewhat and the two shear tests do not compare favorably"

Louisiana obtains its deep samples by hiring a drilling contractor with a rotary drilling rig having a core barrel of 4 in inside diameter and approximately 36 in long Mr Lehmann reports that this method is very satisfactory

Mr Allen reports that Kansas uses two types of drive tube samplers

"One of the samplers has a tube which is split in halves and hinged at the top The two halves are held together by means of a special collar This tube has been used for taking samples of moderate depths (20 to 30 ft) and has been successful except where the soil has been in a very soft saturated condition (saturated silt) having little cohesion and a comparatively low density

"A second type of sampler consisting of a tube which is open only on the cutting end (except for a small air vent) has also been used

"The only advantage gained by the use of the split tube is ease of removing the sample with the minimum disturbance The advantage of a tube with a cutting edge properly tapered and a thin side wall is greater ease in driving, less disturbance to the layer as well as to the soil density The disadvantage is, of course, reduced strength

"The disadvantage of the split tube is in withdrawing the sample, especially in soils which are low in cohesion, or which are very wet and soft

"It is felt that in order to remove very wet samples we must resort to the use of a suction pump in order to obtain a partial vacuum on the top of the sample Obviously this cannot be done with a split tube The use of a tube which has a slightly elliptical bore would aid in shearing off the sample by turning the tube with the initial lift The tube should have a minimum wall and cutting edge thickness to prevent disturbance It should also have a minimum inside diameter of four and one-half (4½) in and preferably six (6) in The greater the diameter of the core the more nearly is it possible to obtain a minimum disturbance in the four and one-quarter (4¼) in diameter sample for the consolidation test Also, the depth of the sample should be at least three (3) times that required for the sample for consolidation"

Iowa uses a drive sampler consisting of a 4 in diameter tube 8 in long, made of 14 gage material The tube is beveled inward $\frac{1}{16}$ in on the cutting end, and slips over a driving head at the top It is secured to the driving head by four $\frac{5}{16}$ in by $\frac{1}{2}$ in cap screws A bead, $\frac{1}{8}$ in by $\frac{1}{8}$ in in section and extending the full length of the tube, projects from the outside of the sampler The driving head is provided with a ball valve for the release of air, and is tapped to receive the $1\frac{3}{8}$ in diameter driving rod This device is used in auger drill holes to depths of 25 ft

The advantages of this sampler are listed by Mr Myers as (a) core bit cannot be pulled off, (b) bit is easily removed from head, (c) cores are easily removed from bit, (d) air is given access to bottom of core as the core is being pulled, (e) bits can be replaced at small cost, (f) air valve permits the escape of air above the core as the sampling device is driven The valve closes when the core is pulled

The chief disadvantages of the device are (a) the fact that it will not bring up very plastic soils, and (b) too much time is required in the field to assemble the extension rod Mr Myers suggests that the connection threads should be inside of the rod, so that the workman can see when the rods are butted against each other

California has the most elaborate equipment for drive tube sampling and apparently is doing more of that type of work than other States The "California Sampler," developed under the direction of Mr T E Stanton, Jr and Mr O J Porter, has been described in detail in the Engineering News Record of June 4, 1936, in the Proceedings of the International Conference on Soil Mechanics and Foundation Engineering, June 1936, vol 1, and in other periodicals The details will not be repeated here, since they are well known to all interested in the subject Essentially it consists of a soil sampling tube into which a plug can be inserted at the lower end permitting it to be driven through difficult soil without filling the tube The plug may then be withdrawn and the sampler driven further into the soil The whole equipment is then pulled out of the soil and the sampling tube removed Three sizes of soil samplers are separated, 1-in for hand work up to depths of 50 ft, 2-in for practically all other types of sampling up to depths of 250 ft, and a 5-in sampler is occasionally used where there is considerable coarse sand and gravelly material

Mr O J Porter, Associate Physical Testing Engineer for the State of California Division of Highways, in direct charge of the sampling, states that the equipment is in almost constant use and is proving very satisfactory

The advantages of this equipment may be listed as follows

1 No casing is necessary except when the skin friction is too high and it is impossible to withdraw the sampler This may occur in the case of deep loose sand It has been found necessary in some cases to put the casing down through such material, but below that depth driving may be continued without the casing

2 The sampler is adaptable to all types of penetrable soils, including some not usually sampled by this type of equipment It has been used at the bottoms of churn drill holes, in shales, sandstone and other soft and broken bedrock formations and is considered by Mr Porter superior to core drilling in such material

3 Even in cases where churn drilling or other methods are first used to sink a hole, it is not necessary to clean the hole carefully before lowering the sampler The plugged end can be driven a short distance through the broken soil at the bottom of the hole and the sampler then extended This is a distinct advantage over many other types of samplers where disturbed soil at the bottom of a hole will enter the sampling tube

4 The samples go to the laboratory in the containers in which they were extracted The job of trimming the ends of the cylinders, capping, taping, and covering with paraffin requires a minimum of time and handling and the samples therefore usually reach the laboratory in satisfactory condition

5 The positive air seal above the sample obtained by backing off the plug makes it possible to withdraw nearly any type of plastic soil satisfactorily

6 The cost of operation is relatively low, so that undisturbed samples may be obtained by this method at about the same price as is usually paid for disturbed samples

7 By keeping a driving record, it is possible to determine the variation of resistance to penetration, thus making it necessary to take samples only at points where the resistance changes

Disadvantages

1 Possibly the greatest objection to this sampler has been the size of samples which it obtains From long experience with the equipment Mr Porter thinks the samples are plenty large enough He points out that if larger samples are desired, the 5-in sampler can be used at the bottom of drilled holes, as is ordinarily done with other samplers In general the highway department feels that many feet of small sized samples are more valuable to them than a few feet of larger diameter cores

2 The equipment will not satisfactorily sample clean sand or gravel These materials tend to run out of the bottom on withdrawal and are generally loosened and not in the original form

DISCUSSION

From the examples of practice described, it is evident that there is considerable difference in opinion as to what constitutes satisfactory sampling Circumstances naturally alter cases, and equipment which may be satisfactory in one part of the country may be inadequate in another However, there are two principles of sampling for bearing capacity which should always be kept in mind The first of these is that the prism being explored should be adequately covered, and the second is that the samples should be as nearly as possible in the original undisturbed condition Unfortunately it is impossible to obtain the ideal of complete coverage and perfect samples The question of cost is vital, and increases with the number, depth, and size of samples, as well as with the care with which they are taken and prepared for the laboratory

The decision as to what represents adequate coverage must, of necessity, be left to the engineer, for each job is a problem in itself Thorough exploration is essential Wide variations in density are the rule rather than the exception, even in relatively small areas where the soil is of the same general character as far as mechanical analysis is concerned A single sample, even though very carefully taken, may not be at all representative of conditions throughout the prism In order to meet the requirement of thorough coverage without involving prohibitive costs, Michigan and California have developed the equipment previously mentioned, in which the sampler may be used for measuring resistance to penetration Samples need then be taken only where change in resistance indicates a variation in character or density of the soil Another simple method of logging the subsurface above ground water is to drill 28-in holes with a well drilling rig and send a man down the hole to take penetration readings with a Proctor needle This is of no value in gravelly soil, but is very effective in fine-grained material

The second principle of sampling, that of obtaining good samples, is beset with many difficulties According to those most experienced in this work, it is impossible to transfer the soil from its position in the ground to the testing machine without some disturbance taking place which alters the test results The question of how much disturbance can be permitted and still obtain test results of practical value, is one upon which there is a wide variation of opinion For research work, perfection is desired, and any amount of pains in sampling is justifiable For settlement analysis of large structures on beds of saturated clay, where differential settlement of one part relative to another is critical, it is necessary to have truly representative samples in order to get accurate results For the ordinary highway bridge abutments, much less accuracy is required For highway fills, approximate bearing values for the foundation material are frequently satisfactory Excellent examples of sampling equipment which may be adapted to the problem at hand have been presented by the highway departments of Michigan and California

For accurate sampling no truly satisfactory equipment is yet available Clay samples cannot be cut from beds at considerable depths and transferred to the laboratory testing machines without disturbance Loosely consolidated granular materials are almost certainly compacted by the jarring which occurs in the present methods of driving samplers Clean sand can seldom be brought to the surface in anything like its original form There is need for study and development work on this problem, and it is to be hoped that some of the highway departments will find it of sufficient importance to include it in their research programs

Whatever the type of equipment used, there is one important item which should never be omitted In the words of Mr L C Campbell of the New Mexico State Highway Department, "Equip the sampling devices with men to whom sampling is an important function, rather than with men to whom sampling is merely a lot of work" With such equipment the sampling devices now available are capable of producing samples which are adequate for most present-day highway problems Without such men, dependable samples cannot be produced no matter how fine the equipment

DESIGN OF A CELLULAR COFFERDAM

By Harris Epstein
Designing Engineer, Bureau of Yards and Docks, U S Navy Department

SYNOPSIS

This paper presents the theory for design of a cellular cofferdam consisting of a number of steel sheet pile cells, circular in cross section, and filled with earth The theory is illustrated by a practical design for a specific location showing computations for determining the diameter of the cylindrical cells, the necessity for and the amount of foreign fill required within the cell, the depths to which the sheet piles forming the cells must be driven and the required section of the piles In this case the piles must be 79 5 ft long and driven through 45 ft of tule mud, 10 ft of blue clay and penetrate 3 ft of a 35 ft stratum of yellow clay which rests upon hardpan The excavation inside the cofferdam is carried 54 5 ft below the top of the sheet piles

The cofferdam is a gravity retaining wall and is designed accordingly The overturning moment of the fill outside the cofferdam, the resting moment of the cylindrical cells, pressures at toe and heel of cells, resistance to sliding and horizontal shear are computed and from these the design is made

PROBLEM

An 800-ft long graving dock is to be built at a site whose average soil formation is as given in Figure 1 It is assumed that due to the proximity of existing buildings and the very flat angle of repose of the tule mud, the site of the dock cannot be pre-dredged lower than elevation 90

Dredging to elevation 90 is desirable not only from the standpoint of reducing the pressures on the cofferdam structure in which the dock is to be built, but also to enable floating equipment to be used in constructing the cofferdam

The dock is to be founded on piles A cofferdam free of internal bracing is desired in order to facilitate proper driving of the bearing piles, and the placing of the concrete in the dry

The use of a cellular steel sheet pile cofferdam surrounding the site of the dock is indicated The bottoms of the sheet piles forming the cofferdam cells are to be in the yellow clay if possible, but, for reasons of economy, not down to the hardpan, unless found necessary

The original site material in the cells of the cofferdam may be replaced with fill material having an angle of internal friction of 30°.

The characteristics of the soils at the site as determined from laboratory experiments are given in Table 1

TABLE 1

Material	Weight in water, lb per cu ft	Angle of internal friction, deg	Cohesion lb per sq ft
Tule mud	36	25	150
Blue Clay	45	15	420
Yellow clay	60	30	500
Fill (100 lb per cu ft in air)	60	30	400

The maximum allowable interlock stress in steel sheet piles is 6,000 lb per lin inch

It is required to find the diameter of the cylindrical cells, the depths to which the sheet piles forming the cells will have to be driven (depth required both at front and at back), as well as the required section of the sheet piles

The design of the cofferdam becomes unique and somewhat complicated due to the various soil strata of widely differing characteristics through which the cells pass and also to the fact that it is desirable to stop the cells, for reasons of economy before reaching the hard-pan The discussion of the problem may appear lengthy, but for clarity the solution is presented step by step

The cellular cylindrical cofferdam is in fact a gravity wall retaining the earth fill behind it and has to be designed as such The moment of the retained fill around the toe M_0 is the overturning moment, the moment of the weight of the fill inside the cell around the toe of the cell M_1 is the resisting moment The resisting moment must be larger than the overturning moment The ratio of the resisting moment to the overturning moment is the factor of safety against overturning The overturning moment produces pressures on the base which are assumed to vary uniformly from a minimum at the heel to a maximum at the toe Ordinarily the pressure at the toe is limited to a certain maximum value, while the pressure at the heel is

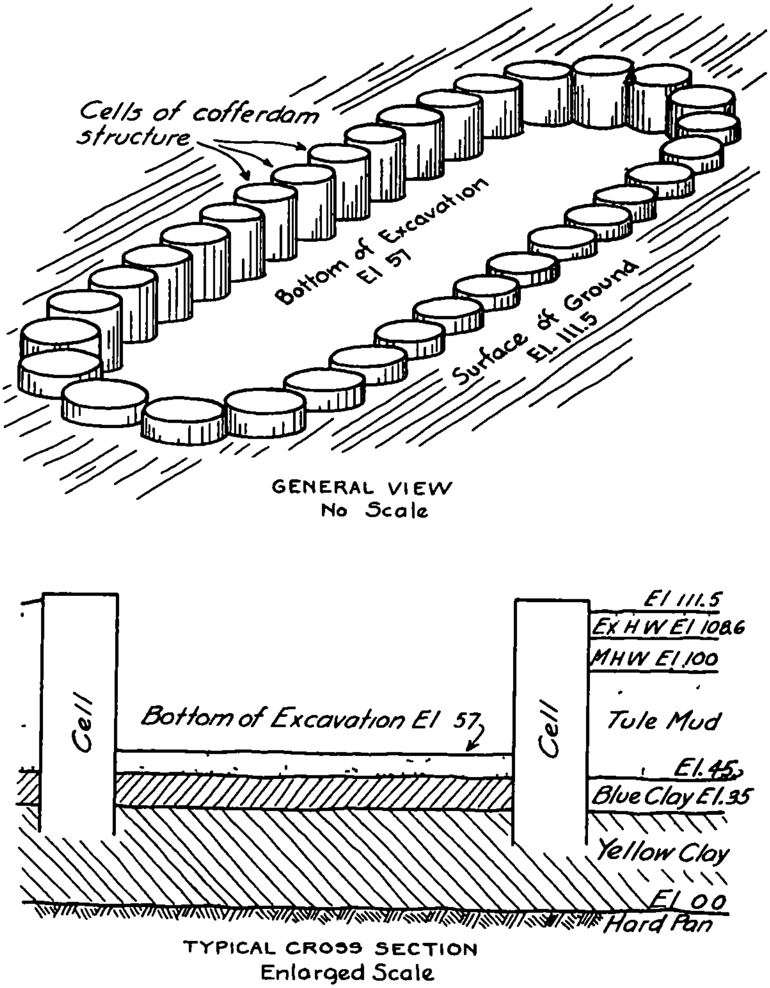


Figure 1

assumed to be 0 The last condition, however, is not a condition *sine qua non* It is true that the soil cannot take any tension but this condition can also be fulfilled with the neutral axis passing through any plane within the circular area

Let R = radius of cylindrical cell
M = overturning moment of the fill around base in kip ft per ft of wall
P = intensity of vertical pressure at bottom of cell per sq ft in kips

Then $M_0 = 2RM$, The overturning moment on one cylindrical cell
$P_0 = P\pi R^2$, Total vertical load inside one cylindrical cell
$M_r = P\pi R^3$, Resisting moment of one cell

$$S_b = \pm \frac{M_0}{I/c} = \pm \frac{8M}{\pi R^2}$$

= unit pressure due to bending

In order that the stress at the heel of the cylinder shall be zero, we must have the intensity of vertical pressure equal to the unit pressure due to bending, thus

$$R = \sqrt{\frac{8M}{\pi P}} = 1\,6\sqrt{e} \tag{1}$$

where

$$e = \frac{M}{P}$$

Factor of safety against overturning

$$f_0 = \frac{M_r}{M_0} = \frac{P\pi R^3}{2RM} = \frac{\pi R^2}{2e} \tag{2}$$

The radius, R, required for a given factor of safety, f_0, is then obtained from

$$R = 0\,8\sqrt{ef_0} \tag{2a}$$

When R is as given by equation (1) we get,

$$f_0 = 4 \tag{3}$$

and maximum compression at the toe in kips per sq ft is

$$P + S_b = 2P \tag{4}$$

The minimum stress at the heel = 0

In order that the cylinder should not slide on the base we must have the resistance to the sliding either equal to or greater than the horizontal force H_0 of the fill behind the cylinder The resistance of cylinder to sliding on the base is made up of the frictional resistance plus the cohesive resistance

Let $\mu = \tan\phi$ = tan of the angle of internal friction of the material at base
and C = coefficient of cohesion in kips per sq ft,
$H_0 = 2RH$

where H = horizontal force of retained fill in kips per lin ft of wall

Then $2RH = \pi R^2(P\mu + C)$

and

$$R = \frac{2H}{\pi(P\mu + C)} \tag{5}$$

Factor of safety against sliding

$$f_s = \frac{\pi R^2(P\mu + C)}{2RH} = \frac{\pi R(P\mu + C)}{2H} \tag{6}$$

The maximum intensity of horizontal shear for a circular beam occurs at the neutral axis and equals

$$v = \frac{Vm}{It} = \frac{4V}{3\pi R^2} = \frac{8H}{3\pi R} \text{ kips per sq ft}$$

where V = total shear for one cell = 2HR

The resistance of material at base in kips per sq ft = $P\mu + C$

Then

$$\frac{8H}{3\pi R} = P\mu + C$$

and

$$R = \frac{0\,85H}{P\mu + C} \tag{7}$$

Factor of safety against horizontal shear

$$f_v = \frac{3\pi(P\mu + C)R}{8H} \tag{8}$$

The radius, R, required for a given factor of safety, f_v, is then obtained from

$$R = \frac{0\,85Hf_v}{P\mu + C} \tag{8a}$$

Comparing equations (5) and (7) we conclude that the controlling factor in determining the radius of the cylindrical cell is the maximum shear intensity rather than the sliding resistance

Using formula (1) and (8a) we can find the radius of the cylindrical cells to satisfy the given conditions However before proceeding with the actual design of the cofferdam, we will derive the expression for horizontal active and passive pressure exerted by a soil possessing an angle of internal friction ϕ and a co-

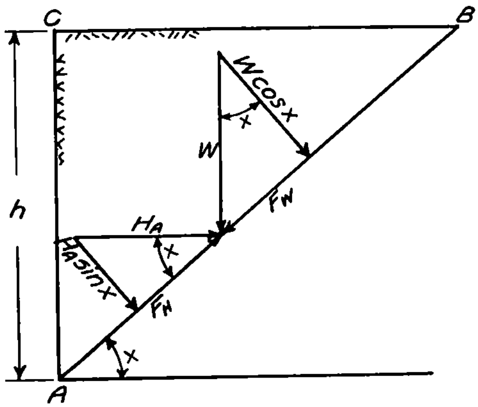


Figure 2

hesive strength of C per sq unit The rupture is assumed to occur along a plane as is done by Coulomb In Figure 1, let AB be the plane of rupture and let x = angle which this plane makes with the horizontal Let also $r = \tan x$ and $\mu = \tan\phi$ The forces acting on this plane are given in Figure 2

Force tending to slide down

$$F_S = F_W - F_H$$

But $F_W = W \sin x$ and $F_H = H_A \cos x$

$\therefore\ F_S = W \sin x - H_A \cos x$

where W = the weight of ABC

$$= \frac{wh^2}{2}\cot x = \frac{wh^2}{2r}$$

and H_A = horizontal force required to stop W from sliding down

w = density of material

The forces resisting sliding F_R = force of friction F_F + resistance due to cohesion F_C

Now $F_F = N\mu$ and $F_C = CA$

Where N = normal pressure on $AB = W\cos x + H_A \sin x$

and $A = \text{Area of } AB = \dfrac{h}{\sin x}$

$$\therefore\ F_R = F_F + F_C = \mu(W\cos x + H_A \sin x) + \frac{Ch}{\sin x}$$

At the point of sliding we have $F_S = F_R$

or
$$W\sin x - H_A\cos x = (W\cos x + H_A\sin x)\mu + \frac{Ch}{\sin x}$$

$$\therefore\ H_A(\cos x + \mu\sin x) = W(\sin x - \mu\cos x) - \frac{Ch}{\sin x}$$

or
$$H_A = \frac{W(\sin x - \mu\cos x)}{\cos x + \mu\sin x} - \frac{Ch}{\sin x(\cos x + \mu\sin x)}$$

Dividing numerator and denominator by cos x we get

$$H_A = \frac{W(\tan x - \mu)}{1 + \mu\tan x} - \frac{Ch}{\sin x\cos x(1 + \mu\tan x)}$$

or
$$H_A = \frac{W(r - \mu)}{1 + \mu r} - \frac{Ch}{\sin x\cos x(1 + \mu r)}$$

Now $\dfrac{\sin x}{\cos x} = r \quad \therefore\ \sin x = r\cos x$

But $\sin^2 x + \cos^2 x = 1$

$$\therefore\ \cos^2 x(1 + r^2) = 1 \quad \therefore\ \cos^2 x = \frac{1}{1 + r^2}$$

and $\sin x\cos x = r\cos^2 x$

$$\therefore\ \sin x\cos x = \frac{r}{1 + r^2}$$

$$\therefore\ H_A = \frac{W(r - \mu)}{1 + \mu r} - \frac{Ch(1 + r^2)}{r(1 + \mu r)} \qquad (9a)$$

Substituting for $W = \frac{wh^2}{2r}$ we get

$$H_A = \frac{wh^2(r-\mu) - 2Ch(1+r^2)}{2r(1+\mu r)} \quad (a)$$

In order to find for what value of r, H will be a maximum we make $\frac{dH_A}{dr} = 0$

or $\frac{dH_A}{dr}$

$$= \frac{wh\mu(1+2\mu r - r^2) + 2C(1+2\mu r - r^2)}{4r^2(1+\mu r)^2} = 0$$

$$\therefore (1+2\mu r - r^2)(wh\mu + 2C) = 0$$

or $\quad 1+2\mu r - r^2 = 0$

and $\quad r = \sqrt{1+\mu^2} + \mu = \tan(45+\phi/2)$

Substituting this value in equation (a) and simplifying we get

$$H_A = \frac{wh^2}{2}\tan^2(45-\phi/2) - 2Ch\tan(45-\phi/2) \quad (9)$$

The weight of sliding wedge is

$$W = \frac{wh^2}{2}\tan(45° - \phi/2)$$

$$\therefore \frac{H_A}{W} = \left(1 - \frac{2Ch}{W}\right)\tan(45-\phi/2)$$

and $H_A = (W - 2Ch)\tan(45° - \phi/2)$ (10)

The intensity of the horizontal active pressure I_A at any depth h from the surface is

$$I_A = \frac{dH_A}{dh} = wh\tan^2(45-\phi/2) - 2C\tan(45-\phi/2) \quad (11)$$

The vertical intensity at the same point $I_V = wh$ The ratio of the intensity of the horizontal active pressure to the vertical intensity is

$$K_A = \frac{I_A}{I_V} = \tan^2(45-\phi/2) - \frac{2C\tan(45-\phi/2)}{I_V} \quad (12)$$

Proceeding in a similar manner for the passive pressure we find that when the wedge of earth is on the verge of moving up

$$H_P = \frac{wh^2}{2}\tan^2(45+\phi/2) + 2Ch\tan(45+\phi/2) \quad (13)$$

or $H_P = (W + 2Ch)\tan(45+\phi/2)$ (14)

The intensity of the horizontal passive pressure I_P at any depth h from the surface is

$$I_P = \frac{dH_P}{dh} = wh\tan^2(45° + \phi/2) + 2C\tan(45+\phi/2) \quad (15)$$

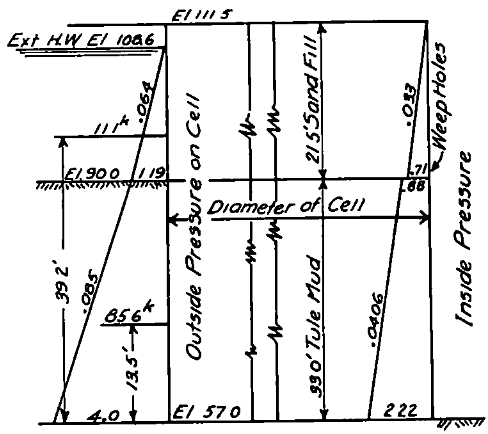


Figure 3

The ratio of the intensity of the horizontal passive pressure to the vertical intensity is

$$K_P = \frac{I_P}{I_V} = \tan^2(45+\phi/2) + \frac{2C\tan(45+\phi/2)}{I_V} \quad (16)$$

DESIGN OF COFFERDAM

Assume cofferdam to stop at elevation 57 and fill inside cofferdam from elevation 90 to surface with a drain at elevation 90 The pressures inside and outside of cells are plotted on Figure 3 according to Table 2 which gives the strength characteristics of the various soils encountered The pressure at rest for the tule mud was determined by laboratory test and found to be the equivalent of a

TABLE 2

Material	φ	tan φ	C—k per sq ft	W in Water K per cu ft	Active pressure						Passive pressure					
	deg				tan $(45 - \varphi/2)$	$\tan^2(45 - \varphi/2)$	Without cohesion		With cohesion		tan $(45 + \varphi/2)$	$\tan^2(45 + \varphi/2)$	Without cohesion		With cohesion	
							I_A	K_A	I_A	K_A			I_P	K_P	I_P	K_P
Tule Mud	25	0 466	0 15	0 036	0 637	0 406	2 015h	0 406	0 015h − 0 192	$0\ 406 - \frac{0\ 192}{I_v}$	1 570	2 464	0 089h	2 464	0 089h + 0 472	$2\ 464 + \frac{0\ 472}{I_v}$
Blue Clay	15	0 268	0 42	0 045	0 767	0 589	0 027h	0 589	0 027h − 0 644	$0\ 589 - \frac{0\ 644}{I_v}$	1 303	1 698	0 076h	1 698	0 076h + 1 094	$1\ 698 + \frac{1\ 094}{I_v}$
Yellow Clay	30	0 577	0 50	0 060	0 577	0 333	0 020h	0 333	0 020h − 0 577	$0\ 333 - \frac{0\ 577}{I_v}$	1 732	3 000	0 180h	3 000	0 180h + 1 732	$3\ 000 + \frac{1\ 732}{I_v}$
Fill (100 lb per cu ft in Air)	30	0 577	0 40	0 060	0 577	0 333	0 020h	0 333	0 020h − 0 462	$0\ 333 - \frac{0\ 462}{I_v}$	1 732	3 000	0 180h	3 000	0 180h + 1 386	$3\ 000 + \frac{1\ 386}{I_v}$

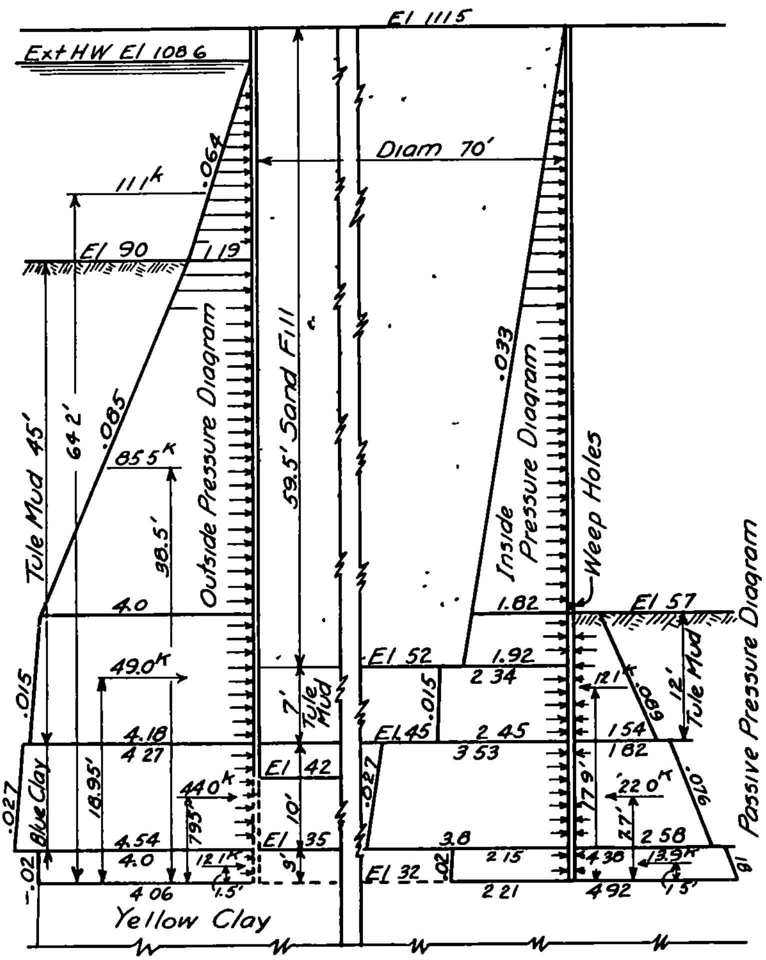


Figure 4

liquid weighing 85 pounds per cubic foot This was used in figuring the pressure behind the cell The pressure inside of the cell was taken as the active pressure In both cases the cohesion was neglected as it is on the side of safety

The forces and their locations are indicated in Figure 3 from which

$$H = 96.7 \text{ kips} \quad M = 1591 \text{ kip-ft}$$

$$P = 21.5 \times 100 + 33 \times 36$$
$$= 3.34 \text{ kips per sq ft}$$

Using a factor of safety against horizontal shear of 1.5 we have from formula 8a and Table 2

$$R = \frac{1.5 \times .85H}{P\mu + C} = \frac{1.275 \times 96.7}{3.34(.466) + .15} = 72.1 \text{ ft}$$

The tension on the interlock, T, of the steel sheet piles at base,

$$T = \frac{IR}{12} = \frac{2.22 \times 72.1}{12} = 13.3 \text{ kips per lin in}$$

This value is more than twice the value allowed, hence we must replace the tule mud inside the cell by the same fill used at the top and make a drain at elevation (57) to reduce the inside pressure The inside and outside pressures on the cell will be as shown on Figure 4

For this case

$$P = 54.5 \times 100 = 5.45 \text{ kips per sq ft}$$

$$e = \frac{1591}{5.45} = 292$$

Using a factor of safety against horizontal shear of 1 5 as before we have

$$R=\frac{1\ 275\times 96\ 7}{5\ 45(\ 577)+4}=35 \text{ ft}$$

For R=35 ft we have Factor of safety against overturning (formula 2) $f_0=6\ 6$ Maximum compression under toe

$$=P+\frac{8M}{\pi R^2}=8\ 76 \text{ kips per sq ft}$$

Maximum tension on interlock

$$T=\frac{1\ 8\times 35}{12}=5\ 3 \text{ kips per lin in}$$

At the allowable 6 kips per linear inch we have allowable $I=2\ 06^{K}/\text{sq ft}$

The actual I due to fill is 1 8, the difference therefore is $2\ 06-1\ 8=\ 26^{K}/\text{sq ft}$ which allows for a head of water inside the cell of $\frac{26}{064}=4\ 0'$ in case the drain does not act temporarily

However, if we stop the sheet piles at elevation 57, the pressure of $8\ 76^{K}/\text{sq ft}$ at the toe will cause the bottom of the excavation to blow up In order to prevent this blowing we have to drive piles in the front to such a depth as to make I_A, the active intensity of the horizontal pressure from inside the cell at that depth, at least equal to the passive intensity in front of the cell at the same point However, before determining the depth to which the front piles will have to be driven, we will determine how far below elevation 57 the fill inside the cell will have to be carried We can not stop it at 57 for in that case the intensity of the active pressure immediately below 57 in the tule mud is

$$I=\ 406\times 5\ 45=2\ 22^{K}/\text{sq ft}$$

$\therefore$ T=6 5 kips per linear inch, which is more than the allowable

Assume that we will carry the fill to a distance x below elevation 57 Then $I_V=5\ 45+\ 06x$, 060 being the weight of the fill in water and $I_A=0\ 406(5\ 45+\ 06x)=2\ 22+0\ 024x$ (neglecting cohesion) The passive intensity at x below elevation 57 for tule mud

$$I_P=\ 089x \text{ (neglecting cohesion)}$$

and the resultant intensity

$$I_r=I_A-I_P=2\ 22-\ 065x$$

$$T=\frac{I_rR}{12}=\frac{(2\ 22-\ 065x)35}{12}$$

Substituting for T=6 0 kips per inch, we get

$$x=2\ 5 \text{ ft}$$

We will carry the fill down 5 ft below elevation 57

Now let us assume that we will drive the piles in the front of the cell a distance y in the yellow clay.

Neglecting at present the increase in the toe pressure due to the increased overturning arm, the vertical intensity, I_V, at the toe inside the cell is

$$I_V=8\ 76+5\times\ 06+7(\ 036)+10\times\ 045+\ 06y \text{ (see Fig 4)}$$

or $$I_V=9\ 76+\ 06y$$

Now $$I_A=I_VK_A$$

From Table 2

K_A for yellow clay is $33-\frac{577}{I_V}$

$$I_A=(9\ 76+\ 06y)\ 33-\ 577=2\ 64-\ 02y$$

The vertical intensity, I_V, at the same point on the excavated side is

$$I_V=12\times\ 036+10\times\ 045+\ 06y=\ 882+\ 06y$$

$\therefore I_P=K_PI_V$

From the table we get

$$K_P = 3 + \frac{1\ 73}{I_V}$$

$$I_P = (\ 882 + \ 06y)3 + 1\ 732$$
$$= 4\ 378 + \ 18y$$

In order to be safe against blowing of bottom we must have

$$I_A = I_P$$

or $$2\ 64 - \ 02y = 4\ 378 + \ 18y$$

or $$y = -8\ 7$$

which apparently means that the pile need not be driven in the yellow clay,

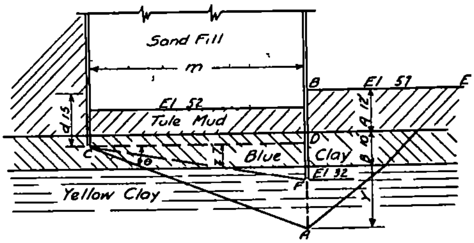


Figure 5

however, if we stop the pile at the yellow clay we get

$$I_A = 9\ 76 \times \ 589 - \ 644$$
$$= 5\ 10 \text{ kips per sq ft}$$
$$I_P = \ 882 \times 1\ 698 + 1\ 094$$
$$= 2\ 59 \text{ kips per sq ft}$$

Therefore I_A is greater than I_P and unless we drive the pile in the yellow clay the bottom will blow Let us drive the front of the piles a minimum of three feet in the yellow clay or to elevation 32 The depth to which the piles in the back of the circular cell will have to be driven is determined by the resistance of the wedge of earth ABE to move up under the active pressure of the earth in back of line AB as shown in Figure 5 Assume the piles in the back to be driven a distance d below the excavation line (elevation 57 0) in the front Let the rupture plane AC make an angle θ with the horizontal so that $\tan\theta = x$ Then by formula 9a we get for the active pressure

$$H_A = \frac{W(x-\mu)}{1+x\mu} - \frac{Ch(1+x^2)}{x(1+x\mu)}$$

μ = tan of the angle of internal friction of the particular strata and W = the weight of the sliding wedge plus the surcharge load P coming on the wedge The surcharge P on line CD is not uniform, but for the purpose of simplicity, it will be taken to be the weight of material above line CD in kips per sq ft Therefore denoting the thickness of any strata by A, its μ by a, its C by C_A and the density by w_a, we have the horizontal base of the sliding triangle

$$b_a = A \cot\theta \text{ or } b_a = \frac{A}{x}$$

and the total sliding weight W_A is

$$W_A = \frac{PA}{x} + \frac{w_a A^2}{2x} = \frac{A}{2x}(2P + Aw_a)$$

and the active pressure for this strata A is

$$H_{AA} = \frac{A(2P+Aw_a)(x-a)}{2x(1+ax)} - \frac{C_A A(1+x^2)}{x(1+ax)}$$

For any other strata of depth B underlying the strata A we have by similarity

$$H_{AB} = \frac{B[2(P+Aw_a)+Bw_b](x-b)}{2x(1+bx)} - \frac{C_B B(1+x^2)}{x(1+bx)}$$

In like manner the value of the active pressure for any other strata can be found

Let the bottom of the last full strata be a distance z below the assumed distance d Denote CD by m Then we get from the figure $Y = mx - z$

Then,

$$H_{AY}=\frac{(mx-z)\,[2(P+Aw_a+Bw_b+Cw_c+\quad)+(mx-z)w_y]\,(x-y)}{2x(1+xy)}$$
$$-\frac{C_Y(mx-z)\,(1+x^2)}{x(1+xy)}$$

The total active pressure of ACD is

$$H_A=H_{AA}+H_{AB}+H_{AC}+\quad+H_{AY}$$

or

$$H_A=\frac{1}{2x}\left\{\frac{A(2P+Aw_a)\,(x-a)}{1+ax}+\frac{B[2(P+Aw_a)+Bw_b]\,(x-b)}{1+bx}+\cdots\right.$$
$$\left.+\frac{(mx-z)\,[2(P+Aw_a+Bw_b+\quad)+(mx-z)w_y]\,(x-y)}{1+xy}\right\}$$
$$-\frac{(1+x^2)}{x}\left\{\frac{AC_A}{1+ax}+\frac{BC_B}{1+bx}+\quad+\frac{(mx-z)C_Y}{1+xy}\right\} \qquad (17)$$

Besides this active pressure there is also the active pressure (*not the pressure at rest*) of the retained material behind the cell above line CD, both of which are resisted by the passive pressure of the wedge ABE in front of the cell

Let ϕ_A, ϕ_B, ϕ_C, ϕ_Y denote the angles of internal friction for stratas A, B, C, Y respectively Let also

$$\alpha=\tan\left(45^\circ+\frac{\phi_A}{2}\right)$$

$$\beta=\tan\left(45^\circ+\frac{\phi_B}{2}\right)$$

$$\sigma=\tan\left(45^\circ+\frac{\phi_C}{2}\right)$$

$$\gamma=\tan\left(45^\circ+\frac{\phi_Y}{2}\right)$$

Then by formula (14) we get the passive pressure for strata A

$$H_{PA}=(W_A+2AC_A)\alpha=\frac{w_aA^2\alpha^2}{2}+2AC_A\alpha$$

For strata B underlying strata A, considering the weight of strata A as a surcharge on strata B, we get

$$H_{PB}=\frac{B\beta^2}{2}(2Aw_a+Bw_b)+2BC_B\beta$$

For strata Y whose thickness is $mx-z$ we get

$$H_{PY}=\frac{C\gamma^2}{2}[2(Aw_a+Bw_b+Cw_c+\ldots)$$
$$+(mx-z)w_y]+2(mx-z)C_y\gamma$$

The total passive resistance is

$$H_P=H_{PA}+H_{PB}+H_{PC}+\quad H_{PY} \qquad (18)$$

The difference between the passive and active pressure is

$$H_r=H_P-H_A$$

H_P is as given above by equation (18) and H_A is as given by equation (17) above, both of which are expressed in terms of the only unknown x That value of x is to be taken which will render H_r a minimum

In our case we will figure the width m of rectangular cofferdam equivalent to the given circular cell as being

$$m=R+K$$

where

$$2KR=\frac{\pi R^2}{2}$$

$$m=R+\frac{\pi R}{4}=1\ 785\times35=62\ 5\text{ ft}$$

For active pressure A=0, B=7 0 ft, while for passive pressure A=12 0 ft; B=10 0 ft

Substituting these values with the weights cohesion and angle of friction for each stratum in equations (17) and (18) we get the net resistance to sliding on the incline

$$H_r = H_P - H_A = \frac{110x^5 + 615x^4 + 700x^3 - 111x^2 + 517x - 28\ 0}{314x^3 + 1\ 7x^2 + 2x}$$

Since the piles in the back and in the front of the cell are stopped at elevation 42 0 and 32 0 respectively, tan θ or x cannot be less than

$$\frac{42-32}{62\ 5} = 0\ 16$$

Substituting this value of 0 16 for x we get

$H_r = 152\ 5$ kips per lin ft of wall

For any value of x larger than 0 16, H_r becomes greater

The horizontal active pressure above line CD (elev 42) is as given in Figure 6 and is $= 11\ 1 + 82\ 3 + 46\ 7 + 12\ 9 = 153\ 0$ kips per lin ft of wall

Hence the factor of safety against sliding $= \frac{152\ 5}{153} < 1$ and it is not safe If we figure the cylinder to slide on line CD (elev 42) the factor of safety against sliding,

$$f_s = \frac{1\ 57(P\mu + C)R}{H}$$

Here H is equal to 153 kips less the passive pressure on BF (see Fig 5) The passive pressure on BF is gotten by substituting for x in equation (18) the value of 16 and is equal to 48 kips per lin ft of wall

$$\therefore f_s = \frac{1\ 57(6\ 14 \times\ 27 + \ 42)35}{153 - 48} = 1\ 09$$

Comparing this factor of safety with the factor of safety for sliding obtained above, we see that critical sliding is the sliding along line CF, the cell however is not safe against sliding and will continue to be so unless we drive the piles in back of the cell into the yellow clay Assume that we will drive the entire cell to elevation +32

Figure 4 shows the pressure diagrams, as well as the total pressures and their locations with reference to bottom of cell for the pressure behind the cell and the passive pressure in front of the cell The overturning moment around bottom of cell (elev 32) is

M = 4894 kip-ft per ft of wall

H = 153 7 kips per ft of wall

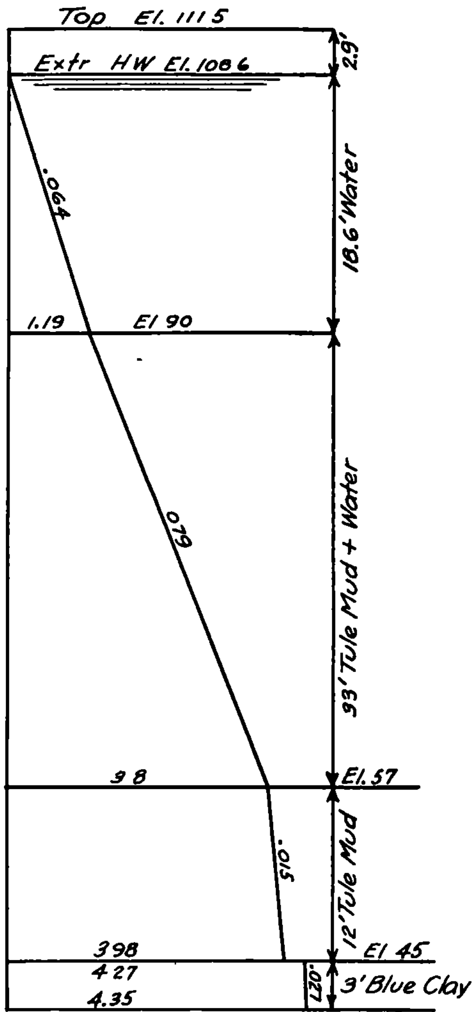


Figure 6

The average pressure on base (elev 32) is

$$P=6\ 65 \text{ kips per sq ft}$$

Now using formulae (2), (6) and (8) we get for factors of safety

$$f_0=2\ 6$$
$$f_s=1\ 6$$
$$f_v=1\ 17$$

The above factors of safety are considered satisfactory considering the temporary nature of the structure

It remains, however, to investigate the sliding of the cell on an inclined plane as well as the blowing of the bottom within the cofferdam

Using formula (17) and making $P=6\ 65$, $A=B=C=z=0$ we get

$$H_A=\frac{171\ 875x^2+695\ 31x-544\ 63}{1\ 16x+2}$$

In formula (18) all symbols used are the same as before except that z becomes now -3 and thus the net resistance to sliding is

$$H_r=H_P-H_A$$
$$=\frac{407\ 81x^3+887\ 65x^2-25\ 31x+640\ 63}{1\ 16x+2}$$

In order to get H_r a minimum we make

$\frac{dH_r}{dx}=0$, which gives $x=186$

Substituting this value of x in the equation for H_r we get

$$H_r=302\ 0 \text{ kips per ft of wall}$$

and $f_s'=\frac{H_r}{H}=\frac{302\ 0}{153\ 7}=1\ 96$ which is safe

The factor of safety against overturning, f_0 is less than 4 and the neutral axis therefore does not pass through the heel of the cell In order to find the maximum pressure on the toe we proceed as follows

The eccentricity of a cell of radius R is

$$e'=\frac{M_0}{P_0}=\frac{2MR}{P\pi R^2}$$

$$\therefore \frac{e'}{R}=0\ 38$$

With this value of $\frac{e'}{R}$ we enter diagram Figure 7, and proceeding according to instructions given thereon we find $b=1\ 2$ and therefore maximum compression at the toe is

$$C_0=\frac{P_0}{bR^2}=17\ 4 \text{ kips per sq ft}$$

In order to make the cofferdam safe against blowing, let us assume that we will drive the piles in the front (at the excavation side) to a depth y below elevation 32 The vertical intensity of pressure at the toe inside of the cell is

$$I_V=17\ 4+\ 06y \text{ kips per sq ft}$$

and $I_A=I_VK_A=5\ 22+\ 02y$

The vertical intensity at the same point on the excavated side is

$$1\ 062+\ 06y \text{ kips per sq ft}$$

and

$$I_P=4\ 92+\ 18y \text{ kips per sq ft.}$$

In order to be safe against blowing we must have

$$I_A=I_P \quad \text{or} \quad y=1\ 9 \text{ ft}$$

This would indicate that the front piles should be driven to elevation $32-1.9$ or to elevation 30 1

If we leave the piles at elevation 32 or make $y=0$ in the above equation for I_A and I_P we get $I_A-I_P=0\ 30$ kips per sq ft which can be overlooked in view of the fact that the maximum pressure of 17 4 kips occurs only at one point

Our completed design is therefore as follows

Radius of each cell$=35\ 0$ ft

All sheet piles are to be driven to elev 32 0 and to have a minimum interlock strength of 12000 lb per lin inch

The fill inside of cells is to be sand weighing 100 lb per cu ft and having an angle of internal friction of at least 30° This fill is to extend from elev 52 0 to elev 111 5 Efficient drains are to be provided at elev 57 0

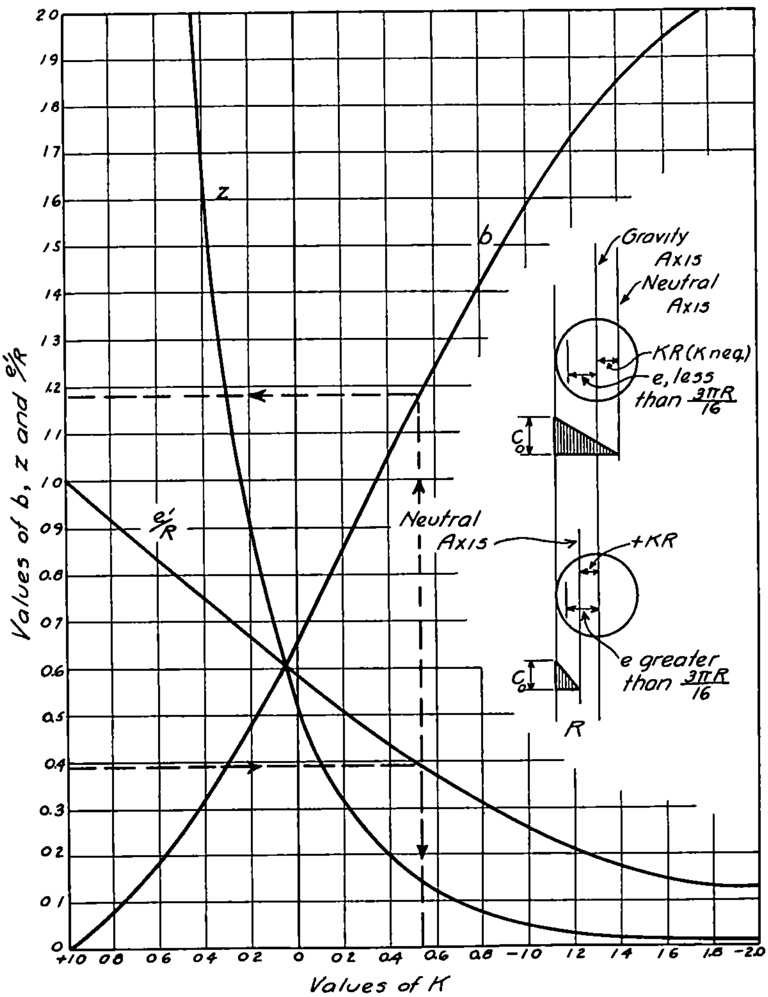


Figure 7

R Radius of circular footing, feet
M_0 Total moment at base of footing, kip-feet
P_0 Total vertical load at base of footing, kips
C_0 Unit pressure on soil (maximum) kips per square foot

1 With M_0P_0 and C_0 known—to find R

$$z = \frac{M_0^2C_0}{P_0^3}$$

From value of z on z curve project vertically to intersect b-curve, reading value of b on vertical scale, substitute value of b in $R = \sqrt{\frac{P_0}{bC_0}}$

2 With M_0P_0 and R known—to find C_0

$$\frac{e'}{R} = \frac{M_0}{P_0R}$$

From value of $\frac{e'}{R}$ on $\frac{e'}{R}$ curve project vertically to intersect b curve, reading value of b on vertical scale, substitute value of b in $C_0 = \frac{P_0}{bR^2}$

3. For location of neutral axis continue vertical projection in either case to K scale

IMPROVEMENT OF SUBGRADES [1]

By Henry C Porter

Engineer of Research, Texas State Highway Department

SYNOPSIS

Because there is greater knowledge of processed surfacing materials than of the natural materials underlying a road, engineers have attempted to design highway pavements strong enough to overcome subgrade defects It is now known that this procedure is impractical, and that no matter how stable the superstructure may be, service qualities will depend largely upon the soil structure beneath Much has yet to be learned about soil mechanics, but many phenomena which are known are still disregarded in subgrade construction Many a pavement failure may be attributed to the placing of granular materials on clays in such fashion that gravity water is impounded Another natural law neglected is in the bulking of damp sand when disturbed and manipulated

The stabilizing of soils by treatment and admixture of other materials is resulting in a great deal of good The methods all have their places and should be used where practical At present, however, it appears that as a whole they are more practical for the roadway superstructure—subbases, bases, pavements and wearing-surfaces—than for underlying soils, especially in high embankments

In the construction of roadways, the engineer must deal with two classes of materials—processed materials, such as cements, asphalts, steels, etc , and natural materials, such as rock, gravel, sand, silt, and clays—both in their natural positions and when disturbed

Until lately, the available information for natural materials has been, in comparison with processed materials, very slight Consequently, design has been rarely thought of as applicable to them Here, an infinite number of variables must be dealt with, the limits of many of which have been known only vaguely, so that mathematical formulas have been unsatisfyingly indefinite Because of this uncertainty, engineers have endeavored to design pavements of processed materials, sufficiently efficient to overcome the defects in the underlying roadway structure composed of natural materials This procedure is now known to be impractical No matter how stable the superstructure is made, its subsequent service will depend to a large extent on the stability of the underlying soil structure—whether it be natural or artificially made

Even though there is a great deal to be learned relative to soil mechanics, there are a great many *known* phenomena that are not yet being generally applied

It is common knowledge that fluctuations of moisture content in soils cause most of the failures Engineers have generally thought that movements in highway soil-substructures were caused by capillary moisture, and that very little could be done in a practical way to prevent it No doubt, in some instances this is a fact, but experiments indicate that in many cases it is permeation of gravity water impounded on clay soils, in addition to their capillary moisture, that causes the trouble

[1] The word "subgrade" as generally used is somewhat indefinite It may refer to a prepared *surface,* or it may refer to a *layer or stratum next under the uppermost principal one,* with no limit as to depth Therefore, in the following discussion, the word "soil-substructure" is used in referring to the man-made part of the roadway structure underlying the subbase, base, pavement or wearing-surface, as the case may be, and the word "subgrade" is used to mean the *prepared surface* of the soil-substructure, for receiving the superstructure, or any part thereof

It can be said with reasonable certainty that in the past more failures have been caused by one single construction detail than all others combined The construction detail responsible for so many so-called "pavement failures" is the placing of granular materials on clays, when flanked by the clays, in such a manner that gravity water is impounded in the comparatively large voids of the granular material The gravity water eventually permeates the underlying clay soil and causes it to become unstable This statement applies not only to the base and subbase materials, but to all the materials in the man-made soil structure

Other known laws of nature that have been applied in the design and construction of the superstructure have been disregarded in the construction of the substructure The bulking of damp sand when disturbed and manipulated is an example In one instance the neglect of this phenomenon in the construction of an embankment in 1926 cost approximately $2500 00 in 1937, for rebuilding and repairing 1200 lineal feet of concrete pavement laid on the unstable soil-substructure in 1930

Another illustration of the subsequent ill effects of bulked sand was illustrated on a project built in 1935 The plans called for a 12-in depth (loose measurement) sand blanket to be placed entirely across the crown of the clay soil-substructure, with concrete pavement to be laid on the sand To permit the contractor to use the original clay shoulders of the roadway for hauling materials, and for operation of the concrete mixer, a 20-ft wide trench was first cut in the top of the clay soil-substructure Sand was placed in the trench to the required 12-in depth, wetted, and the 20-ft width pavement laid on the sand The pavement was cured by ponding water on it After the 10-day curing period, when the clay shoulder soil was cut away to the bottom of the sand under the pavement in order to extend the sand blanket to intersection with the roadway side slopes, air pockets were found between the bottom of the edge of pavement and top of the underlying sand As stated, the sand was wetted after being placed in the trench, but was probably disturbed afterward, during fine grading and shaping operations, and consequently bulked again After the pavement was laid, curing water probably reached the bulked sand, causing it to settle and recede from the bottom of the pavement

At the beginning of the study of irregularities or warping which developed in certain pavement surfaces, it appeared that these conditions occurred after long periods of slow rains and were caused by excessive expansion of clay substructure soil at the places where it was wetted most—such as leaky expansion joints and cracks On page 256, Volume 2 of "Proceedings of the International Conference," held at Harvard University in 1936, data on the subsequent effects of loss of moisture and volume of a soil-substructure are shown During the dry summer following the completion of the project in Navarro County, Texas, moisture and volume were lost to such an extent that the pavement receded in elevation as much as 0 2 ft along the center line and 0 4 ft along the edges Soil moisture content samples taken at different seasons in 1934, 1935 and 1936, showed as much as 14 per cent variation at 12-ft below the ground surface

This phenomenon has been observed in other places in Texas During the unusually dry summer of 1937, investigation of a project near the Gulf Coast revealed the clay soil had receded as much as 0 4 ft below the bottom of the pavement Some of the slabs failed to carry traffic loads, cracked and dropped to the receded soil Most of the cracking was approximately half way between the edge and centerline of the slabs Where one slab receded and the adjoining one did not, uneven joints of as much as 3 in were found

Where highly expansive clay soil must be used for the roadway soil-substructure and the wet and dry seasons of year are extreme, provisions must be made for protecting that soil from being overly wetted during rainy seasons and from undue loss of moisture during droughts This has been accomplished, in some places, by building the shoulders of pervious granular materials to slightly below the bottom of pavement or to the bottom of granular material under the pavement In all cases, the granular shoulder material should extend to intersection with the roadway side slopes This method applies to all types of pavement construction

Some very interesting and conclusive data relative to the accumulation of free water in entrenched sand under pavement are recorded on Test Section No 37, pages 194-198, Part II of the "Compilation of Data on the Guadalupe County Research Project," which experiments were made by the Texas State Highway Department in conjunction with the Bureau of Public Roads On the adjoining Test Section No 36, where the design is the same in every respect, except that the sand blanket extends to intersections with the roadway side slopes, no appreciable free water has ever been found in the sand during the three years of intensive observations

MATERIALS AND DESIGN OF STABILIZED SOIL ROAD MIXTURES

By George A Rahn

Research Engineer, Pennsylvania Department of Highways

SYNOPSIS

Five designs for mixtures of materials found in Pennsylvania based upon cohesion, internal friction and gradation as developed by the researches of the Bureau of Public Roads are presented These mixtures range from fine combinations of soil with sand or screenings to coarser mixtures of properly stabilized soil with graded coarse aggregate Provision is also made for two course work

It is recommended that the shoulders be stabilized with the same mixtures as the traveled way, and that field control of construction be accomplished through frequent analysis of samples, methods for which are given The soil-mortar gradation is the heart of the stabilized soil road, and in all cases should be of the right composition Coarse graded materials are added where practicable but in all cases whether the best coarse material is available or not, the voids in the coarse material should be filled with properly composed soil mortar

This discussion is premised primarily on soil studies and research of the United States Bureau of Public Roads, in which cohesion, internal friction and gradation as laid down by these researches are applied to soils and admixtures such as are encountered in Pennsylvania

The first question which might arise is what materials and procedure fit into the scheme of things? The stabilized soil road is in the low cost field which means that if we are to keep it in this category, local materials must be utilized to the fullest extent

Naturally the first concern is the soil in the road to be improved Is it adaptable to stabilization or would adjacent soil be more economical? By adjacent soil is meant that in the banks adjoining the roadway, or within a short distance of the roadway Also, could it be used in base course construction or could it be used in the top course or is it possible to use it in both bottom and top courses? These questions can be answered by making routine tests on samples from the possible sources of supply

If the soil is fine and plastic in nature what granular material is available which might supply the deficient fractions, is it sand, stone, slag, or gravel screenings? Are there any stone quarries, gravel banks or shale banks in the vicinity or are there industrial waste or mine waste banks close by from which a granular admixture can be secured? Of course if the roadway contains excessive granular material the reverse procedure would have to be applied, in other words, a suitable soil or binder soil secured

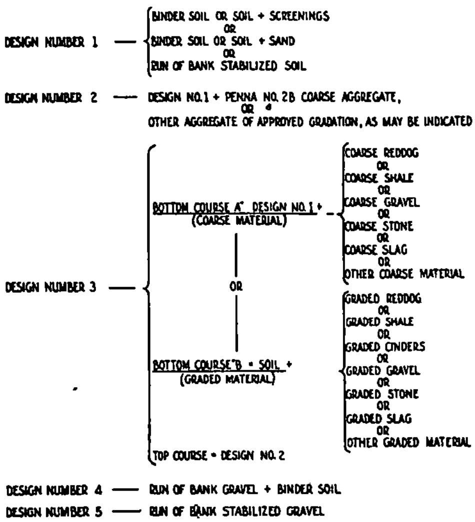


Figure 1 Design of Mixtures, Pennsylvania Highway Department

DESIGN OF MIXTURES

A brief glance at Figure 1 will demonstrate the possibilities of various materials as they fit into the stabilization picture This is a page taken from the Pennsylvania Specifications and headed "Design of Mixtures"

There are five designs covering a wide range of possibilities In these combinations are shown what has been found applicable to construction in the low country, the mountainous country, the industrial and mining countries In other words, we have endeavored to utilize the material available and incorporate it judiciously in order to obtain the best possible results

soil This design can also be regarded as the basic mix, and it will be seen how it fits into the designs which follow

Following is the gradation of the mixture for Design No 1

	%
Passing 1 in screen	100
Passing ⅝ in screen	60-100
Passing ⅜ in screen	50-100
Passing No 4 sieve	40-100
Passing No 10 sieve	30-100

At this point we reach the heart of the stabilized soil road, that is, the actual design of the soil mortar, or the fraction

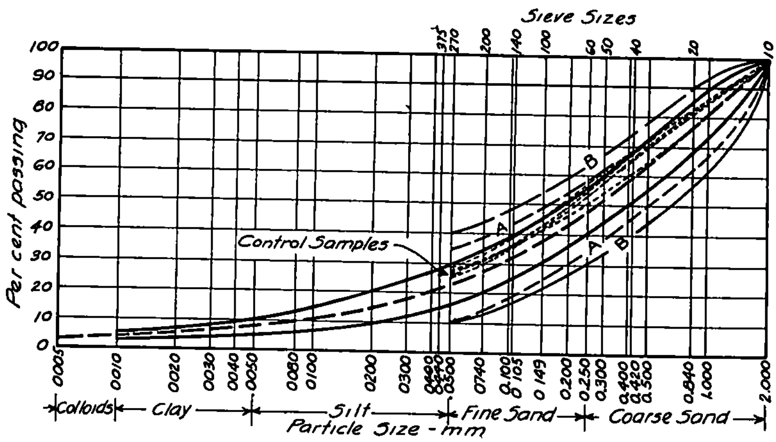


Figure 2 Soil Mortar Gradation Chart

A coordinating factor, namely, that of soil mortar gradation, which comprises the basis of the five designs, will be found in Figure 2

Design No 1 This is composed of binder soil or soil plus screenings, or binder soil or soil plus sand, or run of bank stabilized soil We differentiate between binder soil and soil in this way binder soil is recognized as possessing plasticity, soil does not possess plasticity By screenings we mean the product obtained by crushing stone, slag or gravel, the top size being ⅝-in or ⅜-in down to and including dust Run of bank stabilized soil is a soil possessing the required gradation and plasticity for a stabilized

passing the No 10 mesh sieve, and with this in mind the following gradations have been set up

	Soil Mortar % "A"	Soil Motar % "B"
Passing No 10 sieve	100	100
Passing No 40 sieve	45-70	40-75
Passing No 270 sieve	10-35	10-40

Oversized material is taken care of in the usual manner

In a top course mix this design will have a plasticity index between 0 and 9, and will comply with gradation of soil mortar "A" , in the bottom course mix it will have a plasticity between 0 and 6, and will comply with gradation of soil mortar "B" A liquid limit of not over 35 applies in both cases

Design No 2 This is no more than the addition of a graded coarse aggregate to Design No 1 In the case of Pennsylvania No 2B coarse aggregate this would have a top size of 1¼ in and be retained on the ⅜-in screen, as Design No 1 will consist largely of material passing the ⅝-in or ⅜-in screen The expression "or other aggregate of approved gradation as may be indicated" is to leave it open to trial jobs with aggregate over the 1¼-in size The proportions of Design No 1 mix and coarse aggregate to be combined to form the finished mix for Design No 2 is governed by the amount of the fraction passing the No 10 mesh sieve contained in the Design No 1 mix The finished mix should contain between 30 and 40 per cent of this fraction The plasticity index should be 0 to 9, the liquid limit not over 35 The soil mortar will comply with gradation of soil mortar "A"

Design No 3 This is a two course construction There are two separate designs for the bottom course, which are designated as "A" and "B"

Bottom Course "A" comprises the Design No 1 mix in combination with coarse shale, gravel, stone, slag or other coarse material This coarse material is composed of 2-in to 3-in particles with no appreciable amount of fine material passing the No 10 sieve Taking void content into consideration, a mixture composed of 50 per cent of the coarse material and 50 per cent of the Design No 1 mix are suggested for this design The plasticity index should be from 0 to 9, with the liquid limit not over 35 Soil mortar is to comply with gradation of soil mortar "B"

Bottom Course "B" is composed of soil plus graded shale, cinders, gravel, stone, slag or other graded material Graded material in this case is construed to mean one composed of coarse (maximum size 3 in) and fine particles, with an appreciable amount of fine material passing the No 10 sieve The inclusion of this fine material eliminates the necessity of developing the Design No 1 mix and makes it possible to combine the soil or binder soil directly with the admixture material The plasticity index should be 0 to 6, with the liquid limit not over 35, the soil mortar to comply with gradation of soil mortar "B"

The top course of Design No 3 is similar to that of Designs No 1 or No 2

Designs No 4 and No 5: No 4 represents run of bank gravel combined with binder soil to form the stabilized mix, while the No 5 is a "natural", in other words, it contains all of the characteristics of a stabilized soil The top size for the gravel should not exceed 2¾ in unless used in base course The plasticity index should be between 0 and 9, and the liquid limit 35 The soil mortar is to comply with the gradation of soil mortar "A"

Subgrades When applied to subgrades either Design No 1, Design No 3 bottom course, Design No 4, or Design No 5 may be used and the plasticity index will be between 0 and 12

Shoulders In shoulder construction either one of the five designs may be used, with a plasticity index between 3 and 12

Cross Section Apart from the stabilization of the roadway proper the shoulders are also built of the same stabilized material, dropping from the established depth at the theoretical pavement edge to a feathered edge at the ditch line In conjunction with sound design this cross section provides ease of maintenance

The working out of these various designs in practice is illustrated in Figures 3 to 8 The illustrations start from the fine graded and go through to the coarse graded mixes.

CONTROL

The master gradation chart developed by the Bureau of Public Roads is shown in Figure 2.

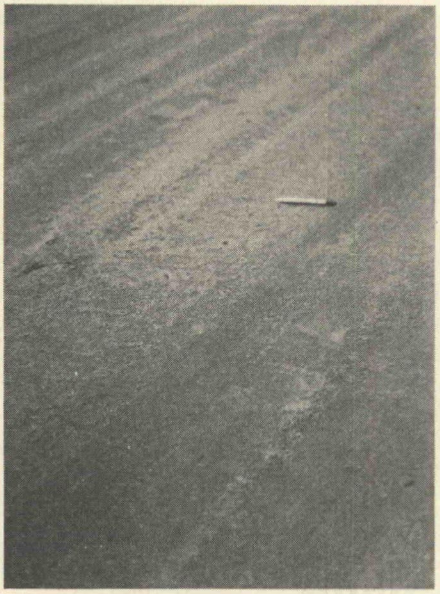


Figure 3. Close Up of a Fine Graded Sand and Soil Mix with a Top Size of Approximately ⅝ In. with 93 Per Cent of Soil Mortar (Passing No. 19 Sieve). This Was Built of Two Materials, a Sand and a Binder Soil Obtained Adjacent to the Road.

Regardless of the selection of materials and their suitability for work of this type, if the job is not satisfactorily constructed poor results can be expected. That control can be exercised is very well demonstrated in Figure 2. Samples taken from the mix at the time of construction are depicted by short dash lines and are designated as "control" samples.

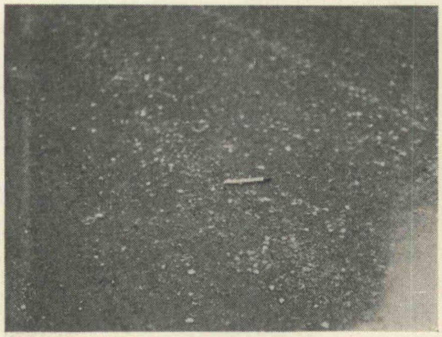


Figure 4. Close Up of a Screenings and Binder Soil Mix with a Top Size of ⅝ to ¾ In. This is a Mixture of Limestone Screenings and Soil.

Such control can be exercised in the field by the washout method, by drying and grading the finished mixture through the coarser screens in the usual manner, while the soil mortar sizes can be determined by

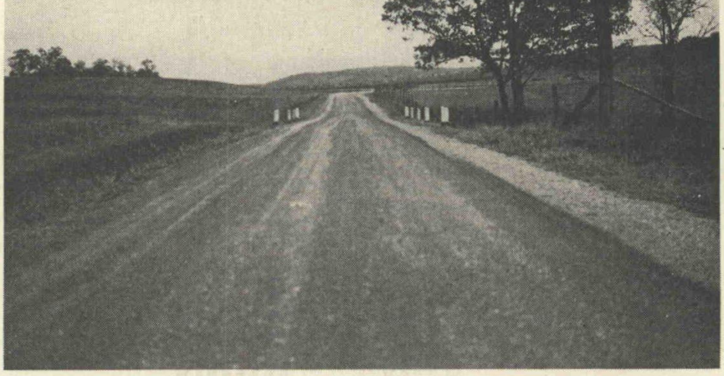


Figure 5. Coarser Mix, Top Size Approximately 1¾ In. Run of Crusher Slag, Including Fines, Was Incorporated with the Soil in the Roadway

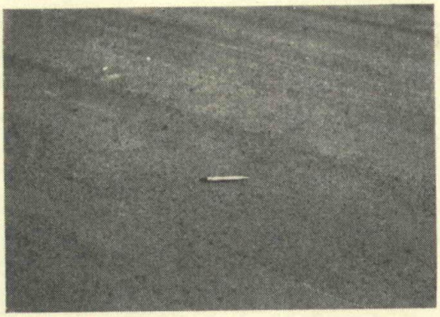


Figure 6. This Is a Close Up of the Surface in Figure 5

dried and sieved over the No. 10 sieve, the lumps of soil and soil adhering to the stone being broken down with the fingers. A predetermined quantity is then weighed out in a tin pan on a small and inexpensive balance. This is soaked in water 15 minutes, then agitated in an egg beater in the presence of water in order to bring about further dispersion, then placed in a metal cylinder with water added to bring the mixture up to a predetermined level, and agitated fur-

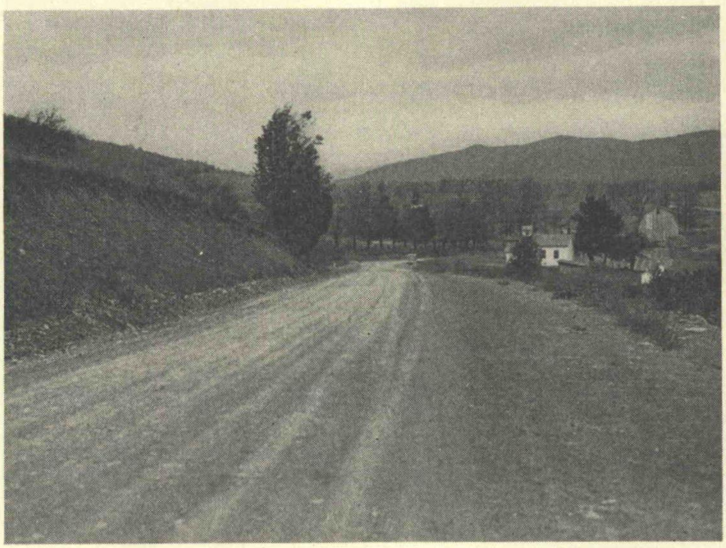


Figure 7. A Coarse Graded Mix with a Top Size of Approximately 2¾ In. In this Case Run of Bank Gravel Was Mixed with the Roadway Soil

washing the mortar through the No. 10, No. 40 and No. 270 mesh sieves, then drying and weighing these fractions, or by means of a hydrometer which has been devised for this purpose. The plasticity indexes are controlled between 0 and 9 since we feel that where restraint is obtained, as in the case of the bottom courses, a zero plasticity can be used satisfactorily. In the wearing surface proper we lean to the lower plasticity index rather than the higher.

In using the hydrometer field control method a sample of the finished mix is ther for approximately one minute. Following this the cylinder is placed on a firm surface and allowed to stand for 1½ min., at which point a metal hydrometer is lowered into the mixture and allowed to stand another ½ min. Then the quantity of No. 270 mesh material present is read directly on the stem of the hydrometer at the top of the meniscus. The temperature of the mixture is taken and the hydrometer reading corrected by a temperature correction chart.

The apparatus and steps in this operation are illustrated in Figure 9. The hydrometer is made of very thin metal and calibrated in the laboratory to read in percentage of No. 270 mesh material present in the mixture.

coarse material where possible. However, at times it is economically impossible to follow the orthodox gradation, as in the case of waste piles, bank run

Figure 8. Representative Surface Five Years Old. The Picture Was Taken in January 1937

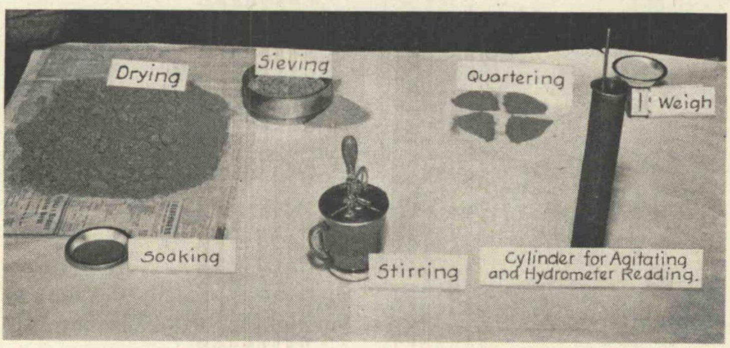


Figure 9. The Hydrometer Method of Determining the Material Finer than the 270 Mesh Sieve

CONCLUSION

In this discussion I have tried to present the basic design; namely, the following out of the gradation prescribed in the soil mortar gradation curve, which insures density and utilization of graded gravels, etc. In all cases see that the soil mortar gradation is properly carried through, insure the correct filling of the voids of the large particles with the stabilized soil mortar and accept the grading as it exists above the soil mortar sizes.

PRINCIPLES OF SOIL MECHANICS INVOLVED IN FILL CONSTRUCTION

By L A Palmer and E S Barber

Division of Tests, U S Bureau of Public Roads

SYNOPSIS

The stability of the fill and the supporting power of the undersoil are important factors in design The stresses in the undersoil caused by the weight of the fill are independent of elastic constants since the problem is one of plane deformation The greatest shearing stress at any point in the undersoil is $\frac{p}{\pi}$ where p is the unit load, and if this does not exceed the cohesion corresponding to the maximum allowable deformation, the supporting power of the undersoil is ample If the cohesion of the undersoil is less than $\frac{p}{\pi}$, it does not follow that failure will result Further study of the supporting power of the undersoil is necessary in this case and for this purpose Prandtl's method of plastic equilibrium may be used The ϕ circle method is applied to the problem of the stability of the fill itself

The design of a fill requires consideration of the stability of the fill itself and the supporting power of the undersoil Seepage, hydrostatic uplift, and capillary saturation are factors which affect the stability of fills, but do not prevent its approximate determination by analytical methods Gradual settlement of the fill, due to consolidation of the fill materials, the undersoil, or both, is also a very important consideration, but this is not necessarily associated with the resistance to shear, upon which stability depends The limitations in design of cross section and the possible factors of safety should be known before construction of the fill, and in this respect soils must be considered in the same light as any other engineering material

STATEMENT OF THE PROBLEM

Fills fail when deformations in the fills or their foundations exceed those permissible [1] in the design of the particular structure

The first task, then, is to make a systematic analysis of stress distribution Next must be determined the resistance of the earth materials to these stresses

Observations of numerous slides have led engineers generally to conclude that the most probable surface of failure in an embankment of fairly homogeneous earth is cylindrical in shape, and the slide is approximately circular in cross section A method of determining the position of the "most dangerous sliding circle" by a laborious graphical procedure, termed the "method of slices," has been published in *Public Roads* (1) [2] Donald W Taylor (2) has suggested application of the so-called ϕ circle method, used by Krey (3), in the consideration of safe bearing loads in foundation problems

Determination of stress distributions in the undersoil begins with the point load formula of Boussinesq, which assumes the stressed material to be isotropic and elastic Integrating the point load formula for all the points located on a straight line furnishes an expression of the stress produced at a point in the earth

[1] The safe working strength of soils, it is emphasized, is not necessarily based on their ultimate strengths For cohesive soils especially, it is more often determined from the stress-deformation relations of the particular soil involved, and is based on the allowable settlements, deformations, or other movements of the structure being designed See Figure 7 Report of Department of Soils Investigations, page 474 this volume

[2] Figures in parentheses refer to list of references at end

by a uniformly loaded surface strip of infinite length and of infinitesimal width Integration of the expression for line loads over a given width furnishes the quantitative expressions for the normal and shearing stresses at a given point beneath the loaded area Such working formulas have been developed by S D Carothers (4), (5), (6)

Coulomb's formula for the ultimate shearing resistance is

$$s = c + p_n \tan\phi$$

where s = shearing resistance, c = cohesion and p_n = normal pressure, all with reference to a unit of area The pressure, p_n, is normal to the plane of shear and ϕ is the angle of internal friction The laboratory procedures for obtaining the values of c and ϕ, are not considered in this paper and p_n is obtained analytically from the known stress distribution and varies over a wide range throughout the supporting earth below the fill In general, its value is much greater than zero and for this reason it is on the side of safety and expediency to be guided by the simple rule

A cohesive supporting soil is considered as safe if its greatest shearing stress does not exceed the cohesion corresponding to the maximum allowable deformation In such a case no further consideration need be given to the problem of bearing capacity of the supporting soil

If, however, the greatest shearing stress beneath the loaded area exceeds the cohesion of the undersoil, further study of its bearing capacity is required and for this purpose Prandtl's method of plastic equilibrium may be used

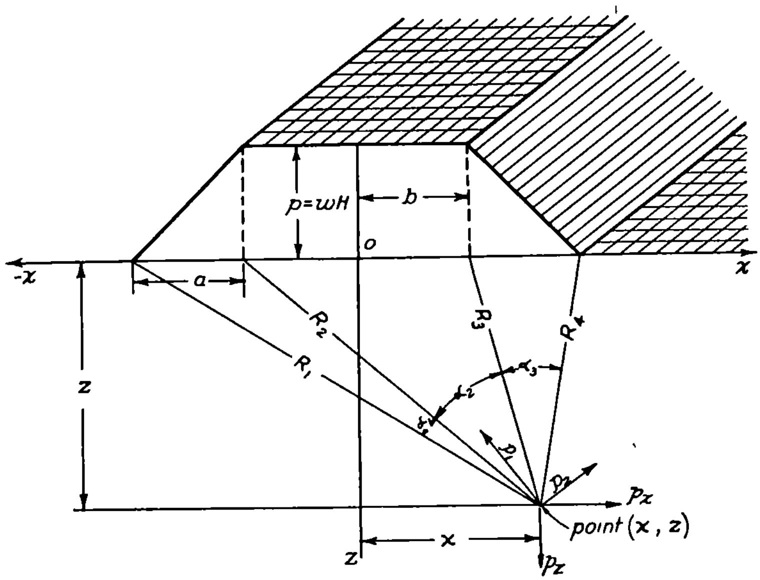
CASE 1 FILL ON GOOD UNDERSOIL

Figure 1 is a pressure diagram of the vertical cross section of a fill The weight of fill material on a square foot at the surface of the supporting soil (excluding the slopes) is wH = p, where w = average weight per cubic foot of fill material and H = height of fill If there is hydrostatic uplift, then w = the buoyed weight of a cubic foot of soil mass Horizontal distance is denoted by x and vertical distance by z The "y" direction is along the length of the fill The problem is one of plane deformation and Figure 1 is in the xz plane The x and z axes and the origin are as shown in the diagram The angles α_1, α_2, and α_3 between the radial lines R_1, R_2, R_3, and R_4, drawn to any point (x, z) in the undersoil, are measured in radians The major principal stress, p_1, is in the direction of the bisector of angle α_2 and the minor principal stress p_2 is perpendicular in direction at any point to p_1 The vertical normal stress at any point (x, z) is p_z and the horizontal normal stress is p_x, the subscripts referring to the direction of these two normal stresses The shearing stress corresponding to p_z and p_x is s_{xz} The maximum shearing stress at a point (x, z) is s max and this is equal to one-half the difference of the two principal stresses at that point, that is $s\ max = \frac{p_1 - p_2}{2}$. The loci of points of constant s max are shown in Figure 2 for the distance "a" equal to the distance "b" (Fig 1) The greatest value of s max for this case is 0 31p It is on the oz axis at a depth z = 3/2a For a = 2b, the greatest value of s max is approximately 0 3p and is on the oz axis, a depth equal to 0 96a, below the bottom of the fill

For any type of fill, within the range of a = b to a = 2b, the greatest value of s max is approximately 0 3p Undersoils which have cohesions equal to or greater than 0 3p furnish ample support for the fill

This quantity, 0 3p, is the first to compare with the shearing strength of the undersoil If this stress is lower than the strength, the answer is evident, if higher, it does not necessarily follow that danger of failure exists If the undersoil has a uniform strength of 0 3p, it is seen that only inside the s=0 3p curve, Figure 2, is the strength exceeded by the stresses This danger zone is confined and is surrounded on all sides by material having these conditions, a plastic zone having been developed, the theoretical stress diagram is no longer correct

For example, let it be assumed that a



$$p_z = \frac{p}{\pi a}\left[a(\alpha_1+\alpha_2+\alpha_3)+b(\alpha_1+\alpha_3)+x(\alpha_1-\alpha_3)\right]$$

$$p_x = \frac{p}{\pi a}\left[a(\alpha_1+\alpha_2+\alpha_3)+b(\alpha_1+\alpha_3)+x(\alpha_1-\alpha_3)-2z\log_e\frac{R_1R_4}{R_2R_3}\right]$$

$$s_{xz} = -\frac{zp}{\pi a}(\alpha_1-\alpha_3)$$

$$p_1 = \frac{p}{\pi a}\left[a(\alpha_1+\alpha_2+\alpha_3)+b(\alpha_1+\alpha_3)+x(\alpha_1-\alpha_3)-z\log_e\frac{R_1R_4}{R_2R_3}\right]+\frac{zp}{\pi a}\sqrt{\log_e^2\frac{R_1R_4}{R_2R_3}+(\alpha_1-\alpha_3)^2}$$

$$p_2 = \frac{p}{\pi a}\left[a(\alpha_1+\alpha_2+\alpha_3)+b(\alpha_1+\alpha_3)+x(\alpha_1-\alpha_3)-z\log_e\frac{R_1R_4}{R_2R_3}\right]-\frac{zp}{\pi a}\sqrt{\log_e^2\frac{R_1R_4}{R_2R_3}+(\alpha_1-\alpha_3)^2}$$

$$s\ max = \frac{zp}{\pi a}\sqrt{\log_e^2\frac{R_1R_4}{R_2R_3}+(\alpha_1-\alpha_3)^2}$$

Figure 1 Stresses in Earth Below Fill

a reserve of resisting capacity The material in the plastic zone may yield to some indeterminate extent and transmit to the adjoining material that part of the load which it cannot resist itself Under fill 20 feet high is to be constructed on the undersoil, Table 1

The value p is equal to wH=90×20 =1800 lb sq ft The greatest shearing stress at any point in the undersoil is

TABLE 1
ASSUMED PROPERTIES OF SOILS

Material	c = cohesion	ϕ = angle of internal friction	w = weight per cubic foot
	lb per sq ft	*Deg*	*lb*
Fill	200	5	90
Undersoil	800	15	110

$0.3p = 0.3 \times 1800 = 540$ lb per sq ft, which is less than 800 lb per sq ft, the cohesion of the undersoil Furthermore, the total shearing resistance at a point on a plane of maximum shearing stress is

$$s = 800 + p_n \tan 15°,$$

where p_n is the pressure that is normal to the plane By inspection it can be seen that p_n is greater than zero and therefore the shear strength furnished by $p_n \tan 15°$ is additive to the shear strength of 800 lb per sq ft furnished by the cohesion alone

In the design of the fill by the "method of slices" it is assumed at the outset that the fill slopes at an angle $i = 45°$ with the horizontal See Figure 3 It is further assumed that the fill material is disturbed and not consolidated It is to be designed safe with respect to the cohesion of the fill material (see Table 1) alone This is explained in the following paragraph

In the older method of "slices" the equation for equilibrium is

$$R\Sigma T = R\Sigma N \tan \phi + RLc$$

where R is the radius of the most dangerous circle, T is the tangential stress

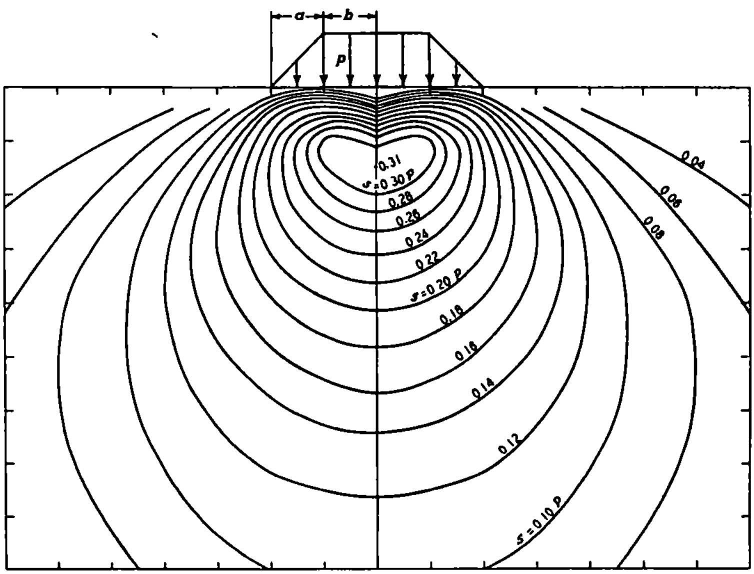


Figure 2 Isoshear Lines Under Typical Linear Fill, a=b

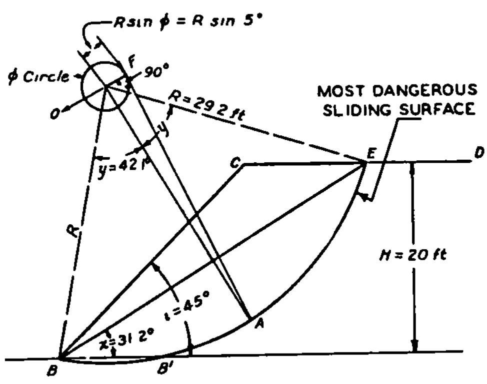


Figure 3 Illustration of the Most Dangerous Sliding Circle for a Given Slope

(tangent to the circle at the midpoint of a given slice), N is the stress normal to the tangential direction, L is the length of the entire arc (of the circle) through the earth, and c is the unit cohesion The expression on the left of the equation of equilibrium, $R\Sigma T$, is the shearing stress moment That on the right, $R\Sigma N \tan\phi + RLc$, is the resisting moment Solving for c,

$$c(\text{computed}) = \frac{\Sigma T - \tan\phi\Sigma N}{L}$$

and

$$\frac{c(\text{as determined from tests})}{c(\text{computed})} = \text{factor of safety}$$

with respect to cohesion alone This procedure assumes utilization of all of the available friction, ϕ in the equation being the limiting value of the angle of obliquity

TABLE 2
DATA ON CRITICAL CIRCLES BY ϕ CIRCLE METHOD
(Taken from Table 1, Stability of earth slopes, by D W Taylor)

i Deg	ϕ Deg	x Deg	y Deg	$\frac{c}{FwH}$
90	5	50	14	0 239
45	0			
	5	31 2	42 1	0 136
	10	34	39 7	0 108
	15	36 1	37 2	0 083
30	10	25	44	0 075
15	5	11	47 5	0 070

The values for the angles x and y are obtained from Table 2 (Table 1 of Taylor's article) The fill section, BCD, (Fig 3) is drawn to scale For $i=45°$ and $\phi=5°$ (Table 1), we find in Table 2 that $x=31\ 2°$ and $y=42\ 1°$ This enables us to draw the chord BE (Fig 3) inclined at an angle $x=31\ 2°$ with the horizontal From the points, B and E, draw the radial lines, R, R, to intersect at an angle, 2y, at the center O, of the ϕ circle, which is on the perpendicular bisector of BE

With R as radius and O as center (see Fig 3), describe the arc, BAE, of the most dangerous sliding circle OA bisects the angle, 2y With O as center and $OF = R \sin\phi = R \sin 5°$ as radius, draw the ϕ circle Draw AF tangent to the ϕ circle

The result of the earlier graphical solutions of the slope problem was expressed as a vector quantity, $\frac{2c}{w}$, c being the unit cohesion and w the unit weight of the material If the length of this vector is divided by any linear dimension such as H, the vertical height, the result, $\frac{2c}{wH}$ is a dimensionless abstract number which, when determined for the most dangerous circle, describes the requirements for stability The form used by Taylor for this abstract number ("stability" number) is $\frac{c}{FwH}$ wherein F is the factor of safety with respect to cohesion alone

It will be observed in Figure 3, that the most dangerous (critical) circle as drawn cuts into the relatively cohesive undersoil below the plane, BB′ In the various publications dealing with the theory of the sliding circle it is always assumed that the soil throughout the entire depth traversed by the most dangerous sliding circle is uniform with respect to both cohesion and friction This assumption serves to simplify an otherwise most complicated problem In the present case this assumption is made only with respect to the fill material, but it is also assumed that the resistance to sliding of the fill material over the surface of the more cohesive supporting soil at the plane boundary, BB′, is approximately the same as that which is assumed for the arc BB′ (Fig 3) Actually, the fill would tend to shear along B′AE and the slide over the surface BB′ would be in the nature of a detritus slide If the value of ϕ for the fill material were much greater than 5°, the value of the angle x would be greater than 31 2°, the value of the angle y would be less than 42 1° and the dangerous circle would not cut into the supporting soil below the plane, BB′

The above construction is permissible when the most dangerous circle passes through the toe of the slope, which is the case when $n=\frac{1}{2}(\cot x-\cot y-\cot i+\sin\phi \csc x \csc y)$ is a negative quantity or zero Here n is the ratio of the distance from the toe of the slope to the intersection of the dangerous arc with the ground surface to the vertical height, H On substituting values from Table 2, $x=31\ 2°$, $y=42\ 1°$, $i=45°$, and $\phi=5°$, it is found that n is negative

The quantity, $\frac{c}{FwH}$, for $i=45°$ and $\phi=5°$ is found from Table 2 to be 0 136 In this case, c = cohesion = 200 lb per sq ft, w = unit weight of fill material = 90 lb per cu ft and H = 20 ft F, the factor of safety, is then found as follows

$$\frac{c}{FwH}=0\ 136=\frac{200}{F\times 90\times 20}$$

and

$$F=\frac{200}{90\times 20\times 0\ 136}=0\ 8$$

With respect to cohesion alone, therefore, the fill is unstable for $i=45°$ and H = 20 ft The value of F may be increased by decreasing either i or H Suppose that a safety factor of 2 with respect to cohesion is desired and that the height, H, must be maintained at 20 ft Then the value $\frac{c}{FwH}$ becomes

$$\frac{c}{FwH}=\frac{200}{2\times 90\times 20}=\frac{1}{18}=0\ 056$$

From curves such as shown in Figure 4, it can be found that for $\phi=5°$ and $\frac{c}{FwH}=0\ 056$, i equals 12°

CASE 2 FILL ON QUESTIONABLE UNDERSOIL

The soil data for this case are given in Table 3 The fill is to be constructed with a 1 1 slope and with dimensions as shown in Figure 5 The factor $\frac{c}{FwH}$ is equal to 0 083 when $\phi=15°$ and $i=45°$ (Table 2)

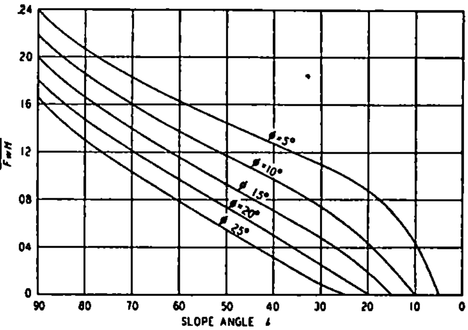


Figure 4 Chart for Stability Number, $\frac{c}{FwH}$

TABLE 3

Assumed Properties of Soils

Material	c = cohesion	ϕ = angle of internal friction	w = weight per cubic foot
	lb per sq ft	Deg	lb
Fill	800	15	110
Undersoil	200	5	100

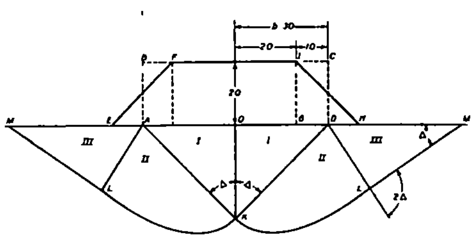


Figure 5 The Surface of Failure in the Supporting Soil

In this case,

$$\frac{c}{FwH}=\frac{800}{F\times 110\times 20}=0\ 083$$

$$F=\frac{800}{110\times 20\times 0\ 083}=4\ 4$$

The fill of itself is, therefore, stable as constructed.

The above computation assumes that *all* of the available friction along the arc of the dangerous circle is utilized in holding the earth in place. If a part of the available friction is not developed or mobilized, there remains a certain unused strength which represents a factor of safety *with respect to friction*. The angle ϕ is the limiting value of the angle of obliquity and if at any point of the circular arc the obliquity of stress is ϕ' which is less than ϕ, then

$$\tan \phi = F_F \tan \phi'$$

where F_F is the factor of safety with respect to friction alone and

$$F_F = \frac{\tan \phi}{\tan \phi'}.$$

Thus for $\phi' = \phi$, F_F becomes unity and for ϕ' less than ϕ, F_F exceeds unity.

The value, 4.4, as above computed, is the factor of safety as related to cohesion alone. Shearing resistance is determined by both cohesion and friction. The true factor of safety, F_t, takes into account both c and ϕ. If the true factor of safety is 4.4, then the cohesion corresponding to this value may be computed and its magnitude compared to the actual cohesion. From the preceding formula, $\tan \phi' = \frac{1}{4.4} \tan 15°$, therefore $\phi' = 3.5°$, the average developed obliquity, or the obliquity which (if constant) yields the same total frictional shearing stress as actually is developed. Taylor has computed the value for $\frac{c}{FwH}$ corresponding to $i = 45°$ and $\phi = 3.5°$ to be equal to 0.147.

Hence

$$\frac{c}{FwH} = \frac{c}{4.4 \times 110 \times 20} = 0.147$$

or c = 1,423 lb. per sq. ft.

This means that to have a *true* factor of safety of as much as 4.4, the cohesion of the fill material must be 1,423 rather than 800 lb. per sq. ft., with only about 23 per cent of the total available friction being utilized.

Next consider the bearing capacity of the supporting soil. With reference to Figure 5, if the supporting power, q, of the undersoil is less than the unit load to which it is subjected, failure takes place over the surfaces MLK, AK, and DK. The diagram represents a section of unit thickness in the direction perpendicular to the plane of the paper. Prandtl's (7) method of plastic equilibrium is applied in this case and in its application we must assume perpendicular slopes. It is on the side of safety to reconstruct graphically the section, EFIH, Figure 5, to have the form of ABCD without changing the volume or mass of fill material. The dimensions of the fill as reconstructed are shown in the diagram (Fig. 5). The width, 2b, is 60 ft., half the width = b = 30 ft. and the height is 20 ft. When the supporting earth fails in shear, zone 1 moves down bodily, shearing at the planes AK and KD. Zone II undergoes a combination of rotation and sliding along the log spiral, KL. Zone III moves outward and upward, shearing along the plane, LM.

The radial line drawn from either of the points, A or D, to any point on the spiral, LK, makes a constant angle 2Δ, with this curve. The value of Δ is $45° - \phi/2$.

It is assumed that the ground water level is at the surface, MM (Fig. 5), of the supporting soil. Taking the weight of a cubic foot of water as 62.5 lb., the buoyed weight of a cubic foot of the supporting soil is 100 − 62.5 or 37.5 lb. = w′ = effective unit weight. The formula used in computing q, the bearing capacity of the undersoil, is

$$q = (c \cot \phi + w'b \cot \Delta) \left[\frac{1 + \sin \phi}{1 - \sin \phi} e^{\pi \tan \phi} - 1 \right]$$

and on substitution of 200 for c, 30 for b, 5° for ϕ and 42 5° for Δ,

$$q = (200 \times 11\ 43 + 37\ 5 \times 30 \times 1\ 091) \left[\frac{1+0\ 087}{1-0\ 087} e^{3\ 1416 \times 0\ 0875} - 1\right]$$

or $$q = 1{,}990 \text{ lb per sq ft}$$

Here e is the Naperian base (natural logarithms) The bearing load per unit area $= 20 \times 110 = 2{,}200$ lb per sq ft The factor of safety, F, is

$$F = \frac{1{,}990}{2{,}200} = 0\ 9$$

This factor of safety may be increased by increasing the width of the fill or by decreasing its height The height of the fill required for a factor of safety of 2 is determined as follows

$$F = \frac{q}{w \times H} = \frac{1{,}990}{110 \times H} = 2,$$

$$H = \frac{1{,}990}{220} = 9 \text{ ft}$$

Supporting Power Is Related to Compressive Strength

It will be of help to the reader of the current literature pertaining to the bearing capacity of soils to bear in mind that the essential features of all bearing capacity formulas may be expressed by the one very simple formula,

$$q = Kp_c,$$

where q = bearing capacity, K is some multiplier, and p_c is the compressive strength of the soil as determined by a laterally unconfined compression test In the derivation of his formula for q, the supporting power, Prandtl used the expression,

$$p_c = \frac{2c \cos \phi}{1 - \sin \phi}$$

which is obtained from the Mohr diagram as shown in Figure 6

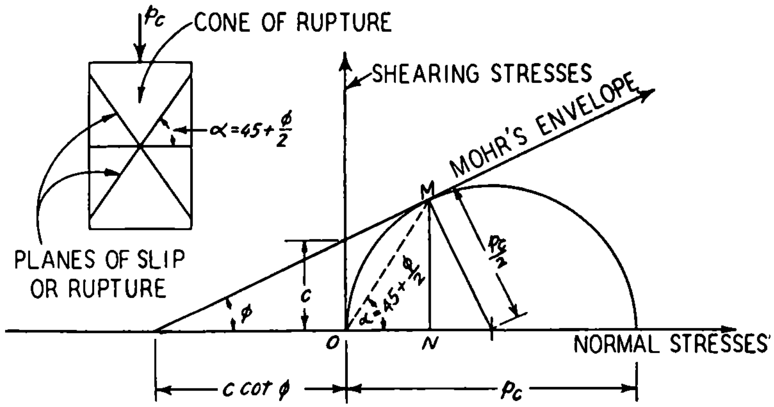


Figure 6 Mohr's Diagram Illustrating Compressive Strength, p_c of a Material *OM* = Plane of Shear, c = Unit Cohesion and ϕ = angle of Internal Friction Shearing Stress on Plane *OM* under Vertical Pressure p_c is *mn*

With a radius equal to $\frac{2}{p_c}$, a circle having its center on the horizontal axis and at a distance $\frac{2}{p_c}$ from the origin O, is drawn For the ideal case the angle α which the planes of slip or fracture of the specimen tested, make with the horizontal, being determined, the line OM is constructed through the origin and at the angle α with the horizontal The envelope line is then drawn tangent to the circle at the point M where the line OM intersects the circle

The intersection of the envelope line with the shearing stress axis discloses, according to Mohr's theory, the cohesion, c, of the material The angle of the envelope line with the horizontal, equals the angle of internal friction of the material, and $\alpha = 45° + \frac{\phi}{2}$

The use of Mohr's diagram in connection with analyses of test data of plastic soils, which deform considerably before failure or have no readily distinguishable planes of slip or fracture, is subject to special considerations not discussed herein

For purely cohesive soil, $\phi = 0$, the values for q in terms of unit cohesion, c, as obtained by the methods of Terzaghi, Krey, and Prandtl, assuming an infinite strip, uniformly loaded, are as follows

Method of Terzaghi, q = 4c
Method of Krey, q = 6 6c
Method of Krey (simplified), q = 6 0c
Method of Prandtl, q = 5 14c

The authors tentatively prefer the method of Prandtl, which is based on well established principles of mechanics as they pertain to the condition of plastic equilibrium For a cohesive soil the method of Terzaghi is more conservative than the others and like Prandtl's method has the advantage of being finally expressed in a single formula The graphical method of Krey does not have this advantage and its use involves considerable time and labor As already mentioned, however, the adaptation of his method to the problem of stability of slopes has simplified the older solution of this problem as presented by Petterson, Fellenius, and others

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DISCUSSION ON SOIL MECHANICS IN FILL CONSTRUCTION

PROF D P KRYNINE, *Yale University* · The authors have applied the general method which is used in such cases, and I wish to say a few words about the method itself The embankment and its foundation are considered in this method as separate bodies whereas in reality they form a whole, and have a common set of principal stresses, although, if the embankment and the foundation are of different materials, there may be some break in principal stresses at the surface of contact In a research project committee of the Highway Research Board, this problem is being considered in some detail

Furthermore, I wish to call attention to the following inconsistency of the so-called "slice method" Vertical pressure at a point located at a certain depth below a slope is measured by the weight of the corresponding earth column If the height of the embankment is increased by placing some additional earth at its top, vertical pressure at the given point computed according to the "slice method" would not change, and this seems illogical

The designer of an embankment should keep in mind that trajectories of maximum shearing stress under the center of the embankment are purely theoretical in character In reality, earth material at that section is confined by the principal stresses and shear action may take place only close to the edges of the foundation

Finally, I believe that the safety factor, 2, which Mr Palmer has assumed, could be somewhat decreased without affecting the stability of the embankment

CEMENT-SOIL STABILIZATION

By W H Mills

Testing Engineer, South Carolina State Highway Department

SUMMARY * OF A SYMPOSIUM PRESENTED AT THE SEVENTEENTH ANNUAL MEETING, HIGHWAY RESEARCH BOARD

Studies of stabilization of soil with cement have advanced in 1937 with the construction of test sections by a number of highway departments and with the continuation of research by the Portland Cement Association The information which is presented here was obtained from reports submitted by·

M D Catton, Development Department, Portland Cement Association
V L Glover, Engineer of Materials, Illinois Department of Public Works and Buildings
T R Perry, Bituminous Engineer, Iowa State Highway Commission
M D VanWagoner, State Highway Commissioner, and Mr W S Housel, Research Consultant, Michigan State Highway Department
F V Reagel, Engineer of Materials, Missouri State Highway Comm
Guy H Larson, Assistant Materials Engineer, State Highway Commission of Wisconsin
W H Mills, Testing Engineer, South Carolina State Highway Department

With minor exceptions methods developed by the Portland Cement Association were followed by the several States in the preliminary laboratory investigations for determining the quantity of cement for stabilizing the soil The thicknesses of the treatments ranged from 4 to 6 in although most of the work has been 6 in thick The mixed-in-place method using disc harrows, quack grass diggers or road machines was used in all experiments, but a traveling mixing plant was used in Missouri and South Carolina on some work Compaction of the mixture at optimum moisture content was secured with sheepsfoot rollers The final finish was obtained by blading the surface and compacting loose material with smooth wheel or pneumatic tired rollers Density determinations of the finished base compared very favorably with laboratory densities obtained by the Proctor method.

Curing of some type was generally used, but in South Carolina and on a portion of the work in Illinois there was no attempt to retain or supply moisture to the finished base Wet earth, wet straw, special curing paper or bituminous materials have been used. In most sections, however, shrinkage cracking occurred in spite of the curing A bituminous wearing surface was applied to most of the sections soon after construction

The initial 500-ft experiment constructed in South Carolina in December 1933 is still in excellent condition and the four other test sections which were constructed in July 1934 have proved entirely satisfactory One of these sections was removed in 1937 on account of new construction work but the others are still under traffic In the Fall of 1935, the Proctor principle of soil compaction was used for the first time on the 1 5-mile experiment near Johnsonville

PORTLAND CEMENT ASSOCIATION

In 1935 the Development Department of the Portland Cement Association started a laboratory research project to develop basic information on soil-cement mixtures Samples of soil types ranging in classification from A-2 to A-8 were obtained from 16 States

Moisture density relations of raw soil and soil-cement mixtures were first determined, then the durability and stabil-

* The complete papers, which will be published separately, may be secured from the Highway Research Board

ity of mixtures of soil and various quantities of cement was obtained on specimens compacted at optimum moisture content and stored in the laboratory for 7 days prior to the start of the test It has since been found desirable to simulate field conditions more nearly by storing laboratory specimens in air of high humidity

The durability tests were alternate wetting-drying and alternate freezing-thawing The wetting-drying cycle consisted of drying the specimens in an oven at 160° F for 42 hours and immersing them in water for 5 hours The freezing-thawing specimens after complete capillary absorption of moisture were placed in the refrigerator at a temperature of −15° F for 20 hours They were then removed and thawed in the moist room on saturated felt pads for 24 hours Losses in weight and changes in volume were determined Based on results obtained, the soils in this series were grouped into three classifications depending on their actions with cement Work on a few unusually bad subsoils of limited occurrence were not included in these groups

Durability specimens made with soils of Group 1 and with 4 to 6 per cent of cement by dry weight showed negligible losses in 12 cycles of wetting-drying and freezing-thawing Volume and moisture changes in these specimens were also quite small Soils in Group 2 were decidedly hardened with 6 to 8 per cent of cement, losses in the durability test were small and moisture and volume changes were reduced to a minimum The soils in Group 3 generally contained considerable clay but 10 per cent of cement hardened them appreciably and the durability tests showed reasonably low losses and volume change

The relation of the Liquid Limit, Plastic Limit and Clay Content were compared with durability results and the following is quoted from Mr Catton's report

"At the present time, no effort has been made to draw sharp lines between soil characteristics and treatment requirements However, the following general conclusions are justified regarding characteristics of soils falling in Treatment Group 1 or 2

"1 The liquid limit must be below 50
"2 The plasticity index must be below 25
"3 The clay content must be below 35
"4 The percentage of solids at maximum density must be 60 or greater
"5 The soil must possess a "regular" moisture density curve

"If a soil meets the above specifications, it is evident that it can be effectively hardened by the addition of a reasonable amount of cement The cement required to harden the soil effectively will be approximately the same as that producing effective hardening in a similar soil in the same treatment group

"All the laboratory results obtained to date have been most encouraging It has been possible to evolve basic principles governing soil-cement mixtures Their application permits the production of consistent, predictable results Specimens prepared and tested in the laboratory have shown substantial durability when subjected to severe durability tests"

ILLINOIS

The Illinois section near Rockford in Winnebago County was 6000 ft long 18 ft wide and 6 in thick Ten per cent of cement by volume was used Durability tests on samples of soil with this quantity of cement showed very low losses

The project was divided into eight sections varying from 500 to 900 ft in length The average time required to spread and mix the cement, apply the water, and compact and finish the surface was 2 hr for 100 ft The scarifying and pulverizing operations were carried on when there was no actual processing in progress, although these operations together with the curing, turn-arounds, and incidentals increased the total time involved Mixing of dry cement was accomplished with disc harrows supplemented by blading Water was added in two lifts due to the depth of the treatment Sheepsfoot rollers compacted the mix at optimum moisture content

The last 700-ft section was given a surface application after compaction of pea gravel applied at the rate of 25 lb per sq yd The surface was then wetted slightly and compacted with trucks, after which it was rolled with an 8-ton roller Considerable loose gravel remained on the surface after this rolling

The following is quoted from Mr Glover's report

"Within two days after the first two increments were completed, hair checking appeared on the surface and it was supposed that these were caused by the rapid and excessive drying out which resulted from lack of curing, therefore, all other increments were cured for seven days In spite of this, transverse cracks and some hair checking appeared on these increments within three days after completion

"When examined in December 1936, approximately three months after completion, the interval between transverse cracks was about 15 ft on all increments except that covered with gravel, on that increment, the interval was about 30 ft At that time, longitudinal cracking was apparent in only one increment, where a continuous crack, at approximately the centerline, extended through the entire length of the increment, a distance of about 800 ft

"When examined in April 1937, scaling and pitting had developed, but aside from being somewhat rough, the surface was in fair condition In order to protect the surface and to provide a better riding surface, all but 400 ft of the project was given a bituminous surface treatment in August 1937"

The total cost exclusive of cement was $0 281 per square yard

Conclusions as given by Mr Glover are

"(a) Preliminary samples on which the job control data are to be based should never be taken until the grading operations have been completed

"(b) Extreme care should be exercised in taking the samples on which the job control data are to be based The locations at which the samples are taken should be carefully selected and a sufficient number of samples secured to represent satisfactorily the soil types and variations within these types"

IOWA

During the summer of 1937 a project totaling 1 6 miles in length was constructed on Primary Road No 40 in Wayne County by the Iowa State Highway Commission The section chosen for the experiment was thought to be typical of a great many miles of gravel surfaced roads of that state, and the material in the road consisted of a mixture of soil and gravel which contained approximately 15 per cent clay

The base was 26 ft wide and 4 in thick with 10 per cent of cement by weight The cement content was based on determinations of density, strength, wetting and drying, bearing, and moisture losses

Cement was spread in two layers over the pulverized soil from a Buckeye spreader attached to the rear of a truck but other construction operations were similar to those used on other projects of this type

It was noted that care must be exercised in rolling the final surface with a smooth steel tired roller as the soil picks up if the surface is too wet

For curing, the completed base was primed with tar on the morning following completion During construction the temperatures ranged from 90° F to 102° F and an average of 1100 linear feet of completed base was constructed per day of 14 hours A bituminous wearing mat type was applied to the completed base

The cost of the completed base including curing was $0 393 per sq yd

Mr Frank L Davis, who was in charge of the project believes that quack grass diggers are excellent for mixing, that with proper equipment, cement in bulk instead of bags would be profitable, that water should be distributed with power driven equipment to obtain uniform application, that two sheepsfoot rollers should be used so as to compact the mix at the optimum moisture content before the moisture evaporates, that only track type tractors should be used to pull sheepsfoot rollers as pneumatic tired tractors pack the soil unevenly, that compaction planes occurring during final compaction can be removed with a fine spike tooth harrow, and that the base should be sprinkled with water about five hours before application of bituminous curing material

MICHIGAN

This report deals particularly with the laboratory investigation and field control of the experimental project near Cheboygan

Based on the preliminary survey, the soils were classified into ten groups for preliminary laboratory work Subsequent tests showed that four classifications would have been sufficient The tests of Michigan Highway Department and the Portland Cement Association were in substantial agreement although the methods used varied in some respects By comparing the grading of the samples with the ideal grading for maximum density, it was found that variations in density could be predicted from the grading curves, poorly graded material giving lower densities From these tests the conclusion is drawn that variation in texture and grading is so accurately reflected in the compacted density that the routine density tests may be the most practical basis of designing the stabilized mixture

The mixtures were proportioned by absolute volume, the cement content being expressed as a percentage of the absolute volume of soil plus cement The void characteristics of the soil-cement mixtures were determined in order to obtain some practical criterion of design which would accurately reflect the properties of the stabilized mixture, particularly with respect to durability

Durability tests followed the general procedure outlined by the Portland Cement Association The author states that the durability tests indicate that a cement-voids ratio of 15 per cent may produce a mixture as durable as present requirements indicate is essential

Samples compacted in the field agreed much better with the density of the road than the preliminary samples The addition of clay to a very sandy soil in which the organic matter was high improved the durability considerably but whether this improvement was due to additional fines which were needed to increase density or to correcting the acidity was not clearly determined

Final rolling by an ordinary steam roller caused displacement of the top inch which resulted in flaking or breaking away of the top surface

The following statements are quoted from Mr Housel's paper

> "Since completion of the project the road has been subjected to one year of weathering with practically no traffic Several inspections have been made and observations will be continued Some sections are in quite satisfactory condition while others show signs of excessive scaling and disintegration"

Conclusions The experience indicates some rather definite relations based on void characteristics of the soil which may be applied to the design of soil-cement mixtures The cement-voids ratio appears to be a controlling factor in producing a durable stabilized mixture Studies must be made of the physical chemistry of soils to determine the effect of chemical composition including such factors as hydrogen ion concentration

While a thorough investigation of soils should be made preliminary to construction, it appears to the writer that the durability tests can scarcely be considered feasible on regular construction projects and should be replaced by much shorter routine tests Study of moisture-density relations requires much less time and might be supplemented by a compression test or something similar as routine procedure In addition, it appears that the difficulty of representative sampling necessitates control of compaction by field control tests conducted in the field in conjunction with each day's work

MISSOURI

Three experimental projects have been constructed by the Missouri Highway Department

The first, 1 5 miles long, was constructed in the Fall of 1936 in Moniteau County The soil consisted of A-4 loam, A-6 clay and A-7 clay loam The Portland Cement Association recommended 12 per cent cement by compacted volume The road was 22 ft wide and 6 in thick compacted

The pulverized soil was protected from rain with Sisalkraft paper Cement was applied from bags spotted along the road at regular intervals Straw was spread over the completed surface as a protection from freezing and to reduce the moisture loss

Rain delayed construction considerably and the temperature dropped below freezing the night after completion of 12 of the 14 sections

This project was allowed to stand without surface treatment until the middle of the Summer of 1937 During this time no base weakness developed, however considerable surface scaling occurred approaching pot-holes in some spots particularly at "turn-arounds" These holes were fairly successfully hand-patched with soil-cement mixtures A light surface treatment was not effective in correcting the surface defects that developed and a later bituminous drag treatment was necessary to give the section good riding quality

Two miles of cement-soil stabilization in Franklin County were included in an extensive study of soil stabilization in 1937 The subgrade consisted almost entirely of the Union Silt Loam of the A-4 group One-half of the project was constructed by the road mix method and a travelling mixing plant was used for the other Cement was varied from 6 to 8 per cent by compacted volume

Approximately one quarter mile sections could be processed each day with the road mix method Sisalkraft paper was used for curing a section approximately 150 ft long, and the remainder was primed with tar on the day following construction

For the section mixed with the travelling mixing plant the material was first scarified and pulverized The soil was windrowed to the center of the road and protected against inclement weather with Sisalkraft paper Cement was emptied on top of the windrowed material Sufficient water was added in the pug mill to raise the moisture content slightly above the optimum The mixture was discharged from the pug mill directly into the spreading machine which distributed the materials over the undisturbed subgrade Immediately behind the finishing machine the mixed material was sheepsfooted in short stretches Final shaping was done with a motor grader and final compaction was secured with a 7-ton roller

Straw was used for curing although Curcrete was used for experimental purposes Due to hot weather and heavy local traffic, the straw was not effective Where Curcrete was used there was slightly less early checking and cracking but after several weeks there was no apparent difference between the sections

It was found necessary to place a drag treatment on this section because of bad raveling which took place before the seal coat work began The riding surface was wavy due to the short sections finished

Another project 5 miles long in St Clair County was constructed by contract between September 18 and October

15, 1937 The longest section processed in one day was 1600 ft and the averaged length was 1313 ft The soils varied from A-2 with 14 per cent clay to A-6 with 28 per cent clay A considerable portion of the road consisted of the remains of a failed oil mat surface which could not be entirely removed and portions of the bituminous treatment were included in the mix A 14-in gang plow proved to be very satisfactory on this project for loosening the road bed and for mixing

Exclusive of the cost of cement which was $ 025 per square yard for each percent of cement used, the cost of these sections were as follows

	Per square yard
Route 5 Moniteau County	$ 34
Route 100, Franklin	
Road Mix	15
Machine Mix	27
Route 13, St Clair County	20

The following conclusions are quoted from Mr Reagel's report

"The costs as given are reasonable and in the range of what one can expect to pay for a reliable base in the low cost program

"Some surface treatment to provide a wearing course is required before putting the road under traffic

"The results obtained do not appear to justify the extra cost of the machine mixing as carried on here

"In the processing it developed that proper care and provision could eliminate the objectionable conditions that develop on "turnarounds" Another development in processing indicates that the use of gang plows in turning over the material during the discing and mixing operations is more effective in preparation of the material and in uniform mixing than the "Orchard Cultivators"

"With good organization it appears that a complete crew of men and equipment will complete, as an average, one quarter mile per working day"

WISCONSIN

Half of the Wisconsin Project consisting of 3 3 miles in Adams County was built in the Fall of 1936 and half in the early Summer of 1937

The soil on this project is very sandy and uniform in size Durability and strength tests showed that 20 per cent cement was not enough to stabilize this soil, due to a deficiency in fine material and to the presence of an excessive amount of organic matter The addition of clay showed marked advantages and it was decided to add 20 per cent of clay and 10 per cent of cement by dry weight to the section constructed in 1936 and 8 per cent cement to that constructed in 1937

The clay was first placed on the top of the sand, then pulverized and partially mixed with the sand Cement was spread on the surface Mixing was done with quack grass diggers and the mix was compacted at optimum moisture content by a sheepsfoot roller The correct moisture content in this very friable soil was very important With a moisture content of 9 3 per cent the soil was dry and crumbly, with 10 to 11 5 per cent compaction was good, but 12 3 per cent moisture resulted in sponginess and a tendency of the soil to peel and stick to the smooth roller

A short section of road processed with the same cement but with no clay could not be compacted with the sheepsfoot roller, and it was necessary to resort to a cleatless crawler tractor and lighter equipment

In finishing it was found better to shave high spots and waste the material than to try to fill low places

For curing the finished surface was covered with damp sand one inch deep which was left in place for 7 to 10 days The average section processed per day was 513 ft in 1936 and 728 ft in 1937

The following is a quotation from Mr Larson's report

"There was some shrinkage after final compaction of the road, as evidenced by the formation of shrinkage cracks noticed at intervals of approximately 25 ft upon the removal of the sand covering Also, there was some scaling and spalling of the surface attributed to improper finishing and attempting to patch or fill low spots The clay could not be completely pulverized with the equipment available, and small clay balls were apparent in the surface of the road It was, therefore, deemed advisable in this case to protect the surface from abrasion with a light bituminous armor coat The section processed in the fall of 1936 came through the winter in good shape It is rather early to make any comment as to ultimate service behavior of either section"

SOUTH CAROLINA

Since the experiments described in Vol 16, Proceedings of the Highway Research Board, the South Carolina Highway Department has built 18 4 miles of cement stabilized base Approximately 15 3 miles were constructed by contract and the remaining 3 1 miles by the department's forces All of this work has been surfaced with a bituminous wearing course one-half to three quarter inches thick

One project approximately 0 5 miles long was designed to give information on the minimum quantity of cement which would satisfactorily stabilize the red clay typical of soil found in a large area of the State According to usual laboratory tests 9 5 per cent cement by compacted volume was required for stabilization This quantity was reduced to 7 5 per cent for one section and 6 5 per cent for another There have been no failures during the six months the project has been exposed to traffic

In the winter of 1936 a 10-mile project was constructed by contract A travelling mixing plant was used and compaction and finishing were performed as usual The soils varied from almost pure fine sand to clay loam containing 25 per cent clay Eight per cent of cement was used with the sandy soil and 10 per cent with the heavier clay soil The bid prices were $0 495 per sq yd for base and $0 18 for the surfacing

A four mile section on this project was primed with tar in March 1937 and soon afterward "blow-ups" occurred in 23 places These "blow-ups" were characterized in a few cases by cracking and shattering of the base for the full depth for approximately 2 lineal ft, but in most places only the top 2 in were visibly affected Shattered portions of the base were removed and easily patched with soil-cement mixture There has been no recurrence of this trouble nor has it appeared on any other project A failure due to improper construction occurred in a section of the road 0 5 miles long After the base had been surfaced and under traffic for a short time the surfacing shoved and it was discovered that the top of the cement stabilized base was soft to a depth of about 1 in No serious trouble has developed from this failure and it has been necessary to patch only a few square yards

The department recently repaired a 1 8 mile section of bituminous surfaced road in which the top-soil base material was bad The bituminous surfacing was broken by scarifying and included in the soil-cement mixture This project is being observed with great interest as there is a considerable mileage of bituminous surfaced roads which could be repaired if this method proves satisfactory

CONCLUSIONS

Based on the information presented in these reports, it appears that cement-soil mixtures give promise of real merit as a road construction material Studies should be continued both in the laboratory and in the field to improve and simplify the laboratory and field methods now used

Adequate preliminary laboratory investigation prior to construction and positive and accurate control during construction are essential

DISCUSSION ON CEMENT-SOIL STABILIZATION

MR A A LEVISON, *Blaw-Knox Company.* Would equipment that would efficiently and rapidly pulverize the natural soil which is to be treated with the cement stabilization be useful in this type of construction?

MR MILLS There is a great opportunity for improvement in the equipment all the way through from pulverization to compaction During most of the experiment we used equipment that we could get easily and we were not satisfied with any of it

MR LEVISON. That is a very interesting response but it does not quite agree with the reaction from other sources which is to the effect that this type of construction has as one of its chief advantages economy, and the thought has been presented that to load work of that type up with expensive machinery would not be economical unless work was done in exceedingly long stretches, say contracts for 40 or 50 miles

MR MILLS We believe that we must keep the construction cost as low as possible, and expensive equipment might increase the cost too much However I believe it is true that there is an opportunity for other equipment that can compete satisfactorily with the equipment that we now have

SAND-BITUMINOUS STABILIZATION

By H C Weathers

Division Engineer of Tests, State Road Department of Florida

SYNOPSIS

In order to obtain a low cost, year-round pavement in sparsely populated sections of Florida where existing road material was impassable sand or silty sand, the Florida State Road Department began the work of stabilizing these sandy materials with bituminous material

Stabilization of sandy soil has been accomplished by the addition of bituminous material, in proper quantity, to existing road material This is mixed uniformly in place with harrows and graders or a traveling plant mixer, and then compacted with rollers

Stability tests on the sand are made prior to addition of bituminous material, and if necessary the stability is increased by the addition of other fine aggregate, pulverized lime-stone or similar material having greater stability The bituminous material used has been cut-back asphalt and tar

The results obtained have been entirely satisfactory About 314 miles of this type of road are carrying traffic with a very low maintenance cost When the surfaces began to show signs of wear they were converted into bases by application surface treatments In some cases the surface treatments have been necessary in about two years, while other pavements have gone seven years and are still holding up The necessity for treatment depends on the type of bituminous material used and the amount of traffic

Sand-bituminous stabilization is as its name implies stabilizing sand or sandy soil with bituminous material to produce a stable pavement, base for surface treatment or subgrade It is a comparatively new type of construction and one which we feel originated and has been developed in Florida

There no doubt has been a time with all who have the responsibility for constructing roads or streets, when considerable mileage of roads had to be constructed and made serviceable for year round traffic with very limited funds This was the case of the Road Department of Florida in 1930 There existed considerable mileage of sandy grade, located many miles from other suitable road building material with practically no railroad facilities The mileage was too great to consider hauling in standard materials for other types of construction and the people were demanding some kind of pavement as the sand was impassable These conditions are the reason for the Department becoming interested in this type of pavement The necessary equipment was obtained and experimental work begun with State forces This work progressed 2 or 3 years until it had passed the experimental stage before any projects were let to contract

The sand-bituminous stabilization is accomplished by mixing the materials mechanically on the road, with harrows and graders or with a travelling plant

A few requisites for the success of this type of construction are

(1) Warm, dry construction period
(2) Light or medium traffic
(3) Sandy soil
(4) Good drainage

It may well be remembered that the sand-bituminous stabilization surface is not the ultimate solution of road surface regardless of location, available material, or volume of traffic The contention is made, however, that this type of road surface does have merit and utility in sections where the existing road material is predominately sand, where a warm and reasonably dry construction period exists, where the traffic is not above medium density, and where good drainage may be obtained It is not recommended where the above conditions do not exist

The sand-bituminous stabilization work when used as a pavement, so far, has been a marked success in Florida and has passed the experimental stage

When this type of construction was first begun very little preliminary testing of the existing road material was done, the reason being that no one knew what tests, if any, were desirable Tests were made on the gradation of the material, more as a matter of record than anything else, for when a so-called poor gradation was found, there was very little that could be done about it without greatly increasing the cost This would defeat the purpose of a low cost pavement, so we used the material as it existed to learn what effect the poor grading would have on the service life of the pavement We did know that foreign matter was undesirable and attempts were made to remove same Also the presence of clay in appreciable quantities was undesirable as it would cause balls in the mix not coated with bitumen, therefore clay determinations were made and the detrimental content was arbitrarily set at 10 per cent

With these preliminary tests this type of pavement construction went along very satisfactorily for a year or so until finally trouble was experienced, which will be discussed later Due to this trouble is was decided that a preliminary survey of all proposed sand-bituminous stabilized projects was needed

Now when a project is proposed the first step is to have a material survey made by a competent engineer During this survey drainage should be investigated, and if the project is not drained adequately, it must be corrected for drainage before any other work is done This is essential for any type of surface, and especially so for this one Assuming drainage is satisfactory the material survey is then made by obtaining representative samples from the surface to a depth of 8 or 10 in of the existing road material These samples should be taken at intervals not exceeding 500 ft or whenever it is apparent the character of the material changes, either in quality or gradation The samples are analyzed for percentages of silica, clay, loam, silt and foreign matter Sieve analyses are made, and stability tests or bearing values are determined

It is preferable to have a well graded coarse sand, however, if this does not exist, it is not advisable to haul in material to correct the grading, unless it can be obtained locally at a very small cost During our work with this type of construction many different gradations of sand have been encountered and used Some of the sands from a grading standpoint, when studied from a theoretical point of view and compared with ideal gradation for fine aggregate for bituminous mixtures, seemed absolutely unfit for use Some of the sands have contained very little 10 or 80 mesh material, being all one size with percentages at times between 80 and 90 of 40 mesh material We have never made an effort, for reasons of economy, to correct gradations and so far we have, surprisingly, had no ill effects It appears from observations to date that a so called poorly graded sand, if made up of angular particles, will afford sufficient stability, while those with round particles may give trouble

After the tests are completed a study of the results should be made to determine the suitability of the material As a rule loam and silt do not affect the quality of the material, in fact, they aid stability by acting as filler and their presence is advantageous Foreign matter if excessive should be removed A small percentage of clay is not detrimental, although percentages greater than about 10 prevent uniformity of mix by balling Any material containing more than this amount of clay should be considered unfit for use, unless the clay is of such character that it will disintegrate during mixing and not create balls

The test which has proven most beneficial has been the stability or bearing value test which was improvised by the writer to eliminate an unstable condition experienced with some materials The mixture would not set up and harden in a reasonable length of time and also would not carry traffic, especially slow moving traffic or stationary loads It appeared that the mix was too soft to hold up any stationary load and just a few minutes' parking would result in the vehicle sinking into the mix to where it would have to be pulled out During the investigation of the trouble it was thought that the character, shape and gradation of the aggregate in the existing road material was probably the cause Microscopic examination of the material disclosed this fact and it was found necessary that some kind of stability test should be made on this material and the results compared to stability tests made on other materials from projects which had proven satisfactory To make these tests we used, with slight modifications, a machine which had been developed by the U S Bureau of Public Roads, for obtaining bearing values of sub-soils Our machine is similar to the one described in *"Public Roads,"* Vol 6, No 2, April 1925, p 38, except that the lever arm system was changed to a single straight line lever It consists of a four-legged stand on which is mounted a lever arm and a soil bearing cup Shot is run from the funnel mounted on the left of the stand into a bucket suspended by a spring balance, on the end of the lever arm The soil cup rests on a plate which screws into the stand and the cup is 3 in in diameter and 3 in deep The bearing plug is circular and has an area of 1 sq in on the bearing face A small ball bearing is welded to the underside of the lever arm and fits in a socket in the top of the bearing plug When starting the test the whole system is leveled up by screw adjustment under the soil cup and the lever arm is balanced by a counterweight on the right of the lever arm

The procedure is to oven dry the sample to constant weight Take 600 g of the oven dried material, break up all lumps, add 10 5 cc of water and mix uniformly The cup is filled with the material and a bearing plate covering the entire surface is placed in the cup and an initial pressure, by hand, is applied to the material The bearing plate cover is then removed and additional material is added and piled conically above the cup and the plate cover is again placed on the material and hand pressure applied The excess above the cup is then removed and a total pressure of 1200 lb is applied The large bearing plate is then removed and the small bearing plate of 1 sq in area is placed in the center of the cup resting lightly on the surface of the material The lever arm of the machine is then balanced and the constantly increasing load applied by allowing the shot to run from the funnel into the bucket until the pressure on the bearing plate is great enough to upset the stability of the material The load is recorded at failure The load or the stability is in pounds and is calculated by multiplying the scale reading by 4, which is the lever arm ratio of the machine

This method of test has been used about four years and has proved very successful and valuable in determining satisfactory material It is rather difficult to determine the exact stability that will or will not prove satisfactory However, from what work we have done we do know that the stability of the material from the project which gave trouble when tested had a value of 6 lb per sq in, while material from other completed projects upon which the mix was entirely satisfactory had values of 25 lb per sq in or more, therefore, until further research can be made, we have arbitrarily established 25 lb per sq in as a minimum stability At the present time we are conducting stability tests on materials from all proposed sand-bituminous stabilization projects and where stability values are found less than 25 lb per sq in we are recommending the addition, in the proper proportion, of a suitable aggregate available to the project, which has a stability value sufficiently high that when blended and uniformly mixed with the existing road material it will produce a material with a stability of 25 lb per sq in or more When it is found necessary to stabilize, samples of all available materials are submitted to the laboratory, stability values are determined, the proper materials selected from these test results and the correct blend is calculated and checked for stability If suitable local materials cannot be found, then it is necessary to ship in suitable material Pulverized limestone has been found very satisfactory

If it is known that this type of pavement is proposed for a particular project before the road is graded the material survey should be made in advance, and suitable material selected so that during the grading operations the project can be made suitable for this type of construction

The Florida Highway Department has used and experimented with several bituminous materials, those most commonly used being cut-back asphalt, refined coal tar and petroleum tar However, we have one project constructed with Pine Tar Pine Tar is still in the experimental stage and is not included, at this date (1937) in the Department's specifications The tar materials seem to make satisfactory bases when surface treated soon after being constructed

SPECIFICATIONS

Following are the specifications for the bituminous materials

Cut-back Asphalt Shall be a pure liquid bitumen, free from water and other decomposition products, cut-back with naphtha It shall meet the following requirements for physical and chemical properties

	Min	Max
1 Viscosity, Furol at 122° F	70	140
2 Distillation A A S H O—T-52 with the following exceptions Samples distilled shall be 200 cc, the weight of this volume to be calculated from specific gravity at 60° F		
Bulb of thermometer shall be immersed to a point ¼" above the bottom of the flask		
Condenser shall be water cooled		
Distillate shall be collected in graduated glass cylinders		
Distillation shall be stopped at 680° F, and the entire residue shall be immediately poured into standard penetration containers and allowed to cool for further tests		
(a) Distillate (percent by volume) to 302° F	0	5
(b) Distillate (percent by volume) to 437° F	20	30
(c) Distillate (percent by volume) to 600° F	26	38
(d) Distillate (percent by volume) to 680° F		38
3 Tests on residue from above distillation		
(a) Penetration at 77° F	85	100
(b) Solubility in CS_2 (percent)	99.50	100

The penetration of the base asphalt, before cutting back with naphtha, shall be between 85 and 100

The Department reserves the right to change the above should conditions warrant Same would consist in a change of asphaltic content and percent of naphtha, also consistence of the base asphalt

The Department has found this cut-back asphalt to be more satisfactory in most cases than any of the other cut-back asphalts experimented with, although on one project, where the existing road material contained a high percentage of silt, loam and some clay, mixing was affected by a tendency of the material to ball with the oil To eliminate this, or to obtain better results, it was found necessary to alter the specifications by requiring the penetration of the base asphalt to be 100 to 120 This gave a little softer grade of material and one which mixed more readily On the other hand, on some projects where the material was weak in inherent stability a 45-60 base penetration asphalt has been used For the majority of materials, however, it is recommended that a base asphalt of 85 to 100 penetration be used

In this specification you will notice a distillation requirement (percent by volume) to 302° F of 0 to a maximum of 5 In our early work we did not have this requirement and found in extremely hot weather that the mix was not homogeneous but contained numerous balls of fat material, some pure asphalt Upon investigation it was found that the percentage of distillate at 302° F was high, around 17 This meant that the naphtha used in manufacturing the cut-back was so highly volatile at the lower temperature that in hot weather, after the cut-back had been distributed into the existing road material, these light ends of the naphtha were evaporating too rapidly before mixing could be accomplished and causing this balling condition, therefore the distillation requirement at 302° F was added and the maximum limit set at 5 percent This has assisted in eliminating the unsatisfactory condition

TABLE 1

	Coal tar in base		Pitch oil in base		Flux		Mixture	
	Min	Max	Min	Max	Min	Max	Min	Max
Water, percent by volume		2		2		2		1
Float at 89 6°F in sec	150	210						
Specific Viscosity, Engler, 50 cc at 212°F			7 5	11 5				
Specific Viscosity, Engler, 50 cc at 104°F					1 1	3 6		
Specific Viscosity, Engler, 50 cc at 140°F							12	32
Distillation, 1st drop			455°F					
Percent by weight								
To 338°F		1			0	7		2
To 455°F		10						12
To 518°F		15						18
To 572°F		30			25	87		30
To 752°F				20				
Residue at 572°F	70						70	
Total bitumen, percent by weight, soluble in CS_2	78		95				88	
Specific gravity at 60°F /60°F			1 20	1 27				

Coal Tar The coal tar shall be composed of a heavy coal tar base containing pitch oil and fluxed with water gas tar or distillates thereof, or light coal tar distillates

The coal tar base shall be composed of 85 to 97 percent by volume of refined coal tar and 15 to 3 percent pitch oil as directed The ratio of pitch oil to tar shall be as directed by the Engineer, which may vary according to the type of soil, acid soils generally taking a higher percentage of pitch oil The mixture shall be composed of 80 to 93 percent base and 20 to 7 percent flux

The refined coal tar and pitch oil in the base, the flux and final coal tar mixture shall meet the requirements in Table 1

The specific viscosity shall be within 3 points, plus or minus, of the specific viscosity designated by the Engineer between the above limits

Petroleum Tar The Petroleum Tar shall consist of not less than 60 percent of refined petroleum tar base, fluxed to the specified viscosity with a tar material (liquid at 60° F) The material shall have the characteristics of tar and when combined in proper proportion with the road material from the proposed sand bituminous road mix project, and air cured, the mixture shall show a tendency to set within 6 hours, have a firm set in 4 days and hard set in 7 days It shall meet the following requirements for physical and chemical properties

	Min	Max
Eng Spec Vis at 158° F	22	32
Percent Water		1
Total Distillate, by wt		
To 338° F	0	5
To 572° F	15	40
Bitumen, percent sol in CS_2	88	
Specific Gravity at 77° F /77° F	110	

The specific viscosity shall be within 3 points, plus or minus, of the viscosity designated by the Engineer between the above limits

On construction in which coal tar is used, after final rolling and checking of the surface for irregularities, and about one month from the time mixing operations are completed, the finished surface shall be treated with 0 20 gal per sq yd of coal tar of the same quality as used in the mix, and covered lightly with sand to prevent picking up by traffic

On construction in which petroleum tar is used, 0 25 to 0 35 gal per sq yd shall be applied as above described except that the material shall be liquid cut-back asphalt as hereinbefore specified This seal treatment using liquid cut-back asphalt may be applied on construction consisting of asphalt if deemed necessary by the Engineer

QUANTITY OF BITUMINOUS MATERIAL

The amount of bituminous material needed to make a satisfactory job varies with the nature and grading of the material to be mixed with the bituminous material and the kind of bituminous material to be used In this respect, this type of mix follows the theory of all bituminous mixtures, in which the quantity of bituminous material shall be sufficient to coat entirely each individual particle of the mineral aggregate with a slight excess to partially fill voids and secure durability Finer graded material contains more individual particles per unit volume, therefore, there is more surface area to be coated and more bituminous material is required

The surface character of the material also influences the amount of bituminous material necessary, as a rough, rugged surface will require more than a smooth one In view of this fact and also the great changes in gradation of the road materials in different locations, and even on the same project, it has been impossible thus far to create a cut and dried table of quantities necessary for this type of construction

The method used at the present time to determine the amount of bituminous material necessary to produce a satisfactory mixture is a combination laboratory investigation and trial mix in the field Samples of the material are analyzed for quality and gradation in the laboratory and small batches made up From the results of this study the field engineer is furnished with an approximate quantity of bituminous material necessary for good results He is then to use his own judgment after trying a section using the predetermined quantity and from visual inspection of the actual road mix should vary the quantity of bituminous material, if necessary, to accomplish satisfactory results The final mixture, after the last application of bituminous material has been added and the mixture thoroughly mixed, should have a black glossy appearance Any mix having a brownish color is too lean and should be reworked by adding more bituminous material and the mixing continued until the black appearance is produced

On the work completed in this State the bituminous material has averaged from a minimum of 3 5 gal to a maximum of 7 gal per sq yd of 6-in compacted pavement

Table 2 shows some of the typical sand gradings used on active projects and the quantity of cut-back asphalt necessary to produce satisfactory results

Please note as the material changes from coarse to fine the quantity of oil must be increased, which is in line with the theory of finer graded aggregates having more surface area However, sometimes this theory is upset as the quality and shape of the individual particle of the material plays an important part as is illustrated by sample 7 This is a slightly coarser material in that it contains more 10 and less 40 than samples 4, 5 and 6 with the 80 and 200 mesh material being about the same, yet number 7 requires a little more oil

It might be well to caution against the use of too much bituminous material The tendency is to use too much as it makes mixing conditions easier, but failures can be caused just as easily by using too much as too little The character of the failures will of course be different but will be just as difficult and costly to correct Failures from a dry mix or a mix with an insufficient amount of bituminous material will be in the nature of dusting and ravelling away of the surface until holes occur Failures from a fat mix or a mix with an excessive amount of bituminous material will be in the nature of shoving into an irregular riding surface or an unstable condition resulting in rutting by traffic

TABLE 2

Sample No	Project No	Station No	Mechanical analysis of sand				Gal of oil per sq yd mix approx 6 in thick
			Pass 10 Ret 40	Pass 40 Ret 80	Pass 80 Ret 200	Pass 200	
1	802-C	491	55	41 0	3 6	0 5	3 20
2	857	15	38 0	54 6	4 2	3 2	3 85
3	857	295	29 2	65 4	3 4	2 0	4 00
4	857	300	28 6	66 4	2 8	2 2	4 00
5	857	450	28 6	68 2	1 4	1 8	4 20
6	857	470	20 8	74 6	2 4	2 2	4 70
7	857	895	33 0	60 8	3 8	2 4	5 00
8	857	885	25 0	70 2	3 0	2 0	5 50

STABILIZING OPERATIONS

In preparing the existing road material prior to application of bitumen, the grade is plowed to a depth of 2 in below the proposed depth of the pavement and one-half foot from each proposed edge All roots and foreign matter are removed as far as practicable The removal of roots on some projects has been a problem, and as no satisfactory equipment is available for this work, garden rakes, spring tooth harrows and hand picking have proven the most effective methods The grade is then dressed to line and grade as shown on the plans

Two types of distributors can be used, namely, the regular pressure distributor and the distributor-trailer tank-unit type The latter has been developed for this type of construction and is preferable When this distributor is used, the tanks are constructed with flues for heating The trailer distributor is attached behind the tank truck, and each tank load is distributed direct without transfer of loads as is necessary when the pressure distributor is used

In beginning, the most suitable end of the project is selected as a starting point so that the material will not be hauled over the finished pavement

All automotive equipment is preferably equipped with balloon type tires of capacity for transporting the loads through sandy material

A section approximately 3000 ft in length is selected for each mixing and finishing operation The bituminous material is applied in successive applications of approximately 0 35 gal per sq yd until the amount required to finish the mix is approximately 0 5 gal, then the applications are reduced as low as 0 15 or 0 20 to avoid flooding the mix The first operation consists of applying bituminous material at the rate of 0 35 gal per sq yd until approximately 1½ gal have been applied and this is mixed with disc harrows to a depth of 3 or 4 in This depth should be about half of the total thickness required This 3 or 4 in of material is next pushed with a blade into windrows clear of the 20-ft width which the pavement is to occupy Bituminous material is then applied to the base or lower half of the existing road material in the quantity of 0 35 gallon, with mixing operations going on continuously until a brown lean mix is obtained, lean base and richer top The material is then shaped with the blade and the windrowed material pulled back in three operations, each one receiving an application of approximately 0 35 gal per sq yd and mixed to guard against any sand streaks between the base and top After all of the material from the windrows is in place the edges and quarters are plowed to the full depth of the required pavement thickness After plowing, the material is pulled from ½ ft outside the specified width to the center with a 12-ft blade, cutting to within ½ in of the specified depth of the finished pavement at the edge After this operation the final applications of bituminous material are made with 0 15 to 0 2 gal per sq yd, and at the same time mixing operations are carried on with harrows and 12-ft blade When using tar the 12-ft blade is used to obtain a uniform mix and the harrows are used only to cut in each application In this connection the 12-ft blade has proven the most useful equipment for mixing and is used extensively on all mixing operations regardless of which bituminous material is used

The material is rolled back to the edges This insures proper depth and uniform mix at this point During the final mixing a retread mixer and blades are used to insure a uniform, homogeneous mix

The final finish of the surface is obtained with long wheel base graders having 12-ft blades, of light construction with a maximum weight preferably of 8000 lb The wheels should be equipped with steel tires 12 to 14 in in width to eliminate sinking into the mix The wheels should also be equipped with scrapers, and heavy door mats or burlap saturated in a 50-50 mixture of kerosene and old cylinder oil or some other suitable material to prevent the mix adhering to the wheels While the mix is new and soft it is shaped to crown and grade For the final finish the blade is turned to its maximum angle with the edges of the pavement The top of the mold board is leaned forward so that the blade is at its maximum skinning or scraping position The loose material is skinned down to a solid firm mix This is essential, otherwise scabs will result on the pavement surface

After the finishing operation, the pavement is rolled with a 5-ton tandem roller The rolling operations should be carried on early each morning before the pavement becomes warm, or on cloudy days It is continued until a smooth surface is obtained free from all roller marks

The surface is then checked with a template and straight edge for crown and smoothness All irregularities greater than ½ in in 10-ft are corrected by shaving off the high places with a sharp blade

The edges of the pavement are cut with axes and shovels to true line The waste mix from cutting edges is salvaged and stock piled for future use in constructing turnouts, parking places, etc All dry, hard material is buried in the shoulder On this type of pavement when the bituminous material used is of such character that the mix becomes hard and brittle at the surface and will not resist wear without excessive dusting, it is necessary after the finish is obtained to apply a 0 2 to 0 3 gal per sq yd seal coat of the same bituminous material as used in the mix When petroleum tar is used it is necessary to make this seal application of cutback asphalt with a 50-60 penetration base asphalt

Sometime during the final finishing and rolling operations, the project engineer should take sufficient measurements of the pavement to check the thickness These can be made at holes or trenches dug into the pavement at various intervals, or a coring tool can be made out of a 1½ in or 2 in piece of steel tubing with a handle and plunger similar to an automobile pump A tool of this kind is used by the Division of Tests of the State Road Department after the pavement is completed to obtain final measurements of the pavement thickness

A traveling plant is sometimes also used for mixing which eliminates a great deal of the mixing procedure and expedites the work

I should like to mention something new that has recently been invented to facilitate mixing when the blade and harrow method is used The success of this type of pavement depends largely on the mix being homogeneous It requires considerable mixing with numerous types of mixing equipment to obtain this condition Under ideal weather conditions it is not so difficult, but when the weather is cool or cold it is very difficult and if the material is too wet, work is suspended until the mix dries Also during the winter months it is generally late in the morning before the mix is warm enough to accomplish very much

This new tool known as a "Hot Blade" was invented by Mr Frank Bullard of Tampa, Florida, and is claimed to be very effective in producing a homogeneous mix in a much shorter length of time It has not yet been tried very much by the State Road Department but has been used quite extensively on work in Hillsboro County, Florida

It consists of a metal oven bolted to the rear of the grader blade, in which are three oil burners spaced one near each end and one in the center of the oven Inside the oven are perforated baffles to diffuse and direct the heat to the rear of the mold board On the rear of the grader is mounted a 42 gal fuel tank which carries 30 gal of kerosene under air pressure of 75 lb The air tank and air compressor are also mounted on the rear of the grader and the air tank carries 100 lb of air It requires about 35 to 40 gal of kerosene per 10 hours to heat the blade The working temperature of the blade is said to be 350° F The mold board is extended 10 in vertically to prevent the mix from coming over the board into the burners It is claimed that with this hot blade the mixing time and cost is materially reduced Also it seems to dry out the material after rain and prevents long delays and enables earlier mixing in the mornings in winter weather It enables heavier bodied bituminous materials to be used which increases the stability of the mix

The personnel required to construct this type of pavement should consist of the following: One competent engineer and one superintendent familiar and experienced with this type of construction About 30 men including mechanics, tractor and grader operators, roller man and unskilled laborers This organization, of course, is dependent on the size of the unit and the volume of work desired to be accomplished

The State Road Department of Florida has built 314 miles of this type pavement, some of which is 7 years old Approximately 255 miles have been constructed by State forces at an average cost of approximately $0 41 per square yard This cost is for actual construction of the pavement including all materials, equipment and labor It does not include any cost of grading

Several counties and some municipalities have constructed considerable quantities of this type pavement and several of the southern and eastern States have inquired about it and are beginning to use it in their road systems

There has been very little maintenance as yet on this type of pavement constructed by the State Road Department The average maintenance cost has been about $180 00 per mile per year, which has been mostly for shoulder maintenance

This pavement deteriorates on the surface slightly with age The bituminous material appears to lose its life and when this condition begins the pavement is given a surface treatment Some projests require surface treating sooner than others, due mainly to the type of bituminous material used in the mix We have some projects that required treatment within two years after construction, while others were treated at five or six years and some few are now seven years old and treatment has not been necessary This type of pavement makes an excellent base for surface treatment and after such treatment the maintenance cost should not be any more than for other types of surface treated bases

Sand-bituminous stabilization has recently been used on weak, sandy subgrades to stabilize them so that they could be compacted by rolling and would support wheel loads The procedure is the same as hereinbefore stated except that a smaller quantity for bituminous material is used It was found that these low stability sand subgrades, although of Group A-3 material, were causing excessive cracking and warping of the concrete pavements laid on them The stability test indicated a stability of 15 lb per sq in , and the addition of bituminous material, equivalent to 1½ gal per sq yd 9 in loose, uniformly mixed, increased the stability to 40 lb per sq in and produced a sub-grade that could be compacted by rolling and one that would support ordinary wheel loads without appreciable rutting

This type of road has been a life saver in that section of Florida which, generally, borders the Gulf of Mexico and traverses a very sparsely populated country, however, it is a link in one of the main routes from Tampa, around the Gulf Coast to Pensacola The people of this section needed a road of some kind very badly as the deep sand was practically impassable On account of lack of railroad facilities and satisfactory material in this section, other than sand, a higher type of pavement was practically prohibitive Necessity, being the mother of invention, started the investigation and experiment with "Sand-Bituminous Stabilization," which has now passed its experimental stage Now 314 miles of this type pavement have been constructed and many more miles of similar construction are anticipated for the future

GEORGE S. BARTLETT AWARD

The George S. Bartlett Award was established in 1931 by a group of friends of George S. Bartlett, with the purpose of perpetuating the spirit of friendship and helpfulness which he brought into his work in the highway field.

It is conferred annually upon an individual who has made an outstanding contribution to highway progress, the recipient being selected by a Board of Award composed of one representative of each of the following organizations:

The American Association of State Highway Officials
The American Road Builders' Association
The Highway Research Board of the National Research Council

The recipients of the Award have been:

Thomas H. MacDonald	1931
Arthur N. Johnson	1932
James H. MacDonald	1933
Frank F. Rogers	1934
Edward N. Hines	1935
Thomas R. Agg	1936
C. A. Hogentogler	1937

IN MEMORIAM

ALFRED H SWAYNE

Resolution of the Highway Research Board

It is with deep regret that we record the death on April 16, 1937, of Alfred H Swayne, Vice President of the General Motors Corporation, First Vice President of the Automobile Manufacturers Association, and a member of the Executive Committee of the Highway Research Board

A member of this Board but a few months, Mr Swayne was widely known throughout the field of transportation and finance A profound student of these subjects, he had a warm human understanding of the social effects of improved transportation as well as an unswerving desire to see controversy solved through research and fact

His understanding and his readiness to cooperate with others in attaining these objectives will cause his counsel to be missed

Be it resolved, therefore, that these expressions shall be given permanent form on the records of the Highway Research Board and that copies of them shall be sent to his sister, Miss Eleanor Swayne, and to his associates on the Boards of the General Motors Corporation and the Automobile Manufacturers Association

MINUTES OF BUSINESS MEETING

HIGHWAY RESEARCH BOARD

December 1, 1937

The meeting was called to order with Chairman Dickinson presiding

ATTENDANCE

Executive Committee

H C Dickinson, Chairman
F C Lang
A T Goldbeck
C M Upham
Pyke Johnson
T R Agg
Burton W Marsh
W W Mack
R W Crum, Director

Member Organization Representatives

American Association of State Highway Officials
E W James
American Institute of Consulting Engineers
Fred Lavis
American Road Builders' Association
C M Upham
American Society of Mechanical Engineers
J G Bergquist
American Society for Testing Materials
Prevost Hubbard
The Asphalt Institute
Prevost Hubbard
Association of Land Grant Colleges
Anson Marston
Automobile Manufacturers Association
Pyke Johnson
Eno Foundation for Highway Traffic Regulation
C J Tilden
Institute of Traffic Engineers
L W McIntyre
R L Morrison
National Crushed Stone Association
A T Goldbeck
National Paving Brick Association
G F Schlesinger
National Sand and Gravel Association
Stanton Walker
Portland Cement Association
F T Sheets represented by M D Catton
Western Society of Engineers
A N Talbot

Minutes of Meeting of November 20, 1936.

Motion That the Minutes of the meeting of November 20, 1936, be approved as printed in the Proceedings of the Sixteenth Annual Meeting

Adopted

Financial Report The Director presented a Financial Report for the fiscal year ending June 30, 1937, as appended to the Minutes of the Executive Committee meeting of May 20, 1937 The Chairman directed that the Financial Report be accepted

Highway Research Census. The Director reported that the Highway Research Census in cooperation with the American Association of State Highway Officials is being compiled and that initial publication is to be expected in 1938

Dues from Member Organizations The Executive Committee reported that the matter of dues from Member Organizations and additional financial support for the Board is under active consideration but that no recommendation is offered at this time

Report of the Nominating Committee The following report of the Nominating Committee was presented

"Your Nominating Committee consists of H G Shirley, State Highway Commissioner of Virginia, P H Bates, Chief, Clay and Products Division, National Bureau of Standards, J G Bergquist, representing the American Society of Mechanical Engineers, C J Tilden, Yale University, and Stanton Walker, National Sand and Gravel Association The committee is charged with the duty of making nominations for three members of the Executive Committee, each to serve for terms of three years

"The following nominations are offered H C Dickinson, Chief, Heat and Power Division, National Bureau of Standards, T R Agg, Dean of Engineering Iowa State College, and W A Van Duzer, Director of Vehicles and Traffic, District of Columbia

"Respectfully submitted in behalf of Nominating Committee

(Signed) STANTON WALKER,
Chairman"

Motion That the report of the Nominating Committee be approved and that the Director be instructed to cast an unanimous ballot for the nominees

Adopted

Resolution A resolution in memory of Alfred H Swayne was presented by Mr Pyke Johnson

Motion That the resolution in memory of Mr Alfred H Swayne be adopted

Adopted

Motion That letters of appreciation be sent by the Director to those participating in the program of the Seventeenth Annual Meeting and to the officials of the National Academy of Sciences and the National Research Council

Adopted

There being no further business the meeting was duly adjourned

ROY W CRUM,
Director