VOL.
13

HIGHWAY RESEARCH BOARD

DIVISION OF ENGINEERING AND INDUSTRIAL RESEARCH
NATIONAL RESEARCH COUNCIL

PROCEEDINGS
OF
THE THIRTEENTH ANNUAL MEETING
OF THE

National Research Council.
"

HIGHWAY RESEARCH BOARD

Held at Washington, D. C.
December 7–8, 1933

EDITED BY
ROY W. CRUM
Director, Highway Research Board

PART I
REPORTS OF RESEARCH COMMITTEES AND PAPERS

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The functions of the Highway Research Board are To prepare a comprehensive national program for highway research, to assist existing organizations to coordinate their activities therein, and to serve as a clearing house for information on completed and current research

It also conducts special investigations financed by interested organizations, and furnishes unique auspices under which work may be carried on that is scientific in character and impartial in its findings, and which is generally accepted as authentic.

The Highway Research Board is composed of representatives of national organizations interested in the development of the highways of the country Connection with all phases of highway industry is maintained by means of research committees, and contact men appointed from the various state highway departments, universities, colleges and municipalities

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THE TAXATION OF MOTOR VEHICLES IN 1932

BY THOMAS H MACDONALD

Chief, U S Bureau of Public Roads

SYNOPSIS

Motor vehicle owners in the United States paid approximately, one billion dollars in 1932 for the right to operate on the public highways

This sum comes directly from the pockets of motor vehicle owners in the form of registration fees, gasoline taxes, mileage and ton-mile taxes, receipts taxes, personal property taxes, special fees imposed by counties and municipalities and other lesser units of government, and many other direct imposts

In an endeavor to determine for the first time the number and amount of all the various kinds of taxes and fees, and how they are imposed upon the numerous different types of vehicles, a special inquiry was undertaken by the U S Bureau of Public Roads during the summer of 1933, covering all of the 48 States and the District of Columbia The present paper is a preliminary report upon the results of this investigation, the full details of which will be published in a separate report

Organization of the statistical personnel which handled the large mass of figures involved in the survey of motor vehicle taxation was directed from headquarters at Washington One or more special statisticians were employed in each State, all of them being persons familiar with the administrative control of motor vehicles and many of them being present or former employees of State departments These statisticians, often with the assistance of extra helpers, worked in the offices of the motor vehicle administrators and others, handling the original records and transcribing from them the desired data In this way, elaborate data on all State fees and taxes were obtained In addition, the local tax situation—county and municipal fees, and personal property taxation—was covered by means of questionnaires to the proper authorities

All reports were sent to Washington for analysis and compilation, and subjected to careful checking

The inquiry was confined solely to the calendar or fiscal year 1932, and the following comments accordingly apply to that period only

DIVERSIFIED METHODS OF TAXATION

The analysis of the reports which were submitted and a detailed study of the State laws further emphasized a fact already known—that there is great diversity in the methods used by the States in taxing motor vehicles. The greatest lack of uniformity was found in the bases on which payments of registration fees were imposed. In all States the rates were graduated so as to increase with the size of vehicle. In the District of Columbia, however, a fee of $1 was imposed on all gasoline-driven vehicles regardless of type, size or class of service. The methods by which this variation in payments of registration fees were achieved were widely different, and in some States appeared to be unduly complex.

For passenger cars registration fees varying with the weight of the vehicle were most commonly applied. Thirty States used this basis, of these, five included the weight of the passengers. Horsepower, alone or in combination with other factors, was used in 18 States, value or list price in three. In eight States the age of the vehicle or the number of times it had been registered was used as a modifying factor in computing registration fees. In Arizona, California, and Washington, a uniform flat fee was imposed on all passenger cars. In several States combinations of two or more of the factors mentioned above were used.

The same varying bases of taxation were found in the registration of busses. Seating capacity, either alone or in combination with other factors, was used in 27 States, net weight in 15, and gross weight in 9.

In the case of trucks registration fees increasing with the capacity of the vehicle were found in 26 States. In 11 States the fees varied with the gross (loaded) weight of the vehicle, in eight States with net weight, and in three States (Idaho, Pennsylvania, and South Dakota), with chassis weight. In six of the States which imposed weight or capacity taxes a flat fee was charged in addition. In Maryland horsepower only was used as the basis for the registration of pneumatic-tired trucks, and horsepower in combination with other factors was used in Louisiana and Tennessee. In Minnesota the fee was based on the value of the vehicle. The tendency to use more than one basis of taxation was quite as common in the case of trucks as it was with other classes.

Trailers and semitrailers were generally registered on the same basis as trucks, but the rates were usually lower.

In many States distinction was made between pneumatic-tired and solid-tired vehicles, the latter paying higher fees. Farm trucks and those operated only in or near cities paid at reduced rates in a number of States.

In the majority of States vehicles operated for hire were taxed more heavily than those privately operated. This additional taxation was imposed in a number of different ways, including higher registration fees, taxes based on vehicle-mileage, ton-mile or passenger-mile taxes, gross receipts taxes, special license or franchise fees, permit fees, and fees imposed for certificates of convenience and necessity.

A further distinction was frequently made in for-hire carriers according to the class of service Common-carrier trucks and trailers and public-carrier busses, accepting business from the general public and operating on a regular schedule over fixed routes, were generally taxed at higher rates than were carriers operating for hire under contracts or agreements for each trip

Taxicabs and other passenger cars for hire were reported separately by 27 States and the District of Columbia. In the majority of these States higher registration fees were charged against taxicabs than against passenger cars In five States and the District of Columbia additional special fees were imposed

Busses were reported in three classes, school busses, contract and sightseeing busses, and public carriers. Only 16 States reported school busses other than those publicly owned or paying only nominal fees, although a number indicated that such busses were included with other registration classifications

Nineteen States and the District of Columbia made the distinction between public-carrier busses and those operated for sightseeing purposes or as contract carriers The remainder reported revenue busses without distinction as to class of service. There is no question, however, that the great majority so reported were operated as public carriers in either urban or interurban service

The results of the survey indicate that for-hire carriers of property were, in one way or another, recognized as separate classes of vehicles in all States except Connecticut, Delaware, Maine, Nebraska, New Hampshire, New Jersey, New York, and the District of Columbia However, a number of the States which have such regulations failed to report separately the numbers of for-hire vehicles or their payments of fees In all, 35 States reported for-hire carriers in such a manner that they could be included in the tabulations of national totals and averages

Of these 35 States 22 reported both contract and common carrier trucks, and in the case of three additional States, Arkansas, Ohio, and West Virginia, contract and common carriers are included together Nine States reported common carriers separately, but did not segregate contract carriers from those privately owned and operated One State, Massachusetts, reported contract carrier trucks but no common carriers

The data on trailers and semitrailers were less complete, 24 reporting common carriers and 20 reporting contract carriers

ESTABLISHING STATISTICAL BASES FOR COMPARISON

In order to present a clear picture of the amounts of motor-vehicle taxes contributed by vehicles of various sizes, it was necessary to adopt a common basis of classification In the case of trucks, tractor trucks, and trailers it was decided to convert all reported figures given in terms of net, gross, or chassis weight into equivalent rated capacity From a study of published lists giving these weight relations for specific makes and models of vehicles, and also of data furnished by representatives of the motor vehicle industry, conversion tables were prepared which give approximate or average values of rated capacity corresponding to given values of net, gross, or chassis weight With the aid of these tables the desired conversions were made

A scheme of classification was adopted which divides all trucks, tractor trucks, and semitrailers into the following capacity groups

1 —1½ tons and less
2 —Over 1½ tons and less than 3 tons
3 —3 tons and less than 5 tons
4 —5 tons
5 —Over 5 tons

Methods of approximation were devised for making this classification in the case of States for which the data could not be written down directly in this form

Busses were classified on the basis of seating capacity, and factors were developed for converting net or gross weight into seating capacity

In order to determine approximately the amounts contributed in gasoline taxes by the different classes of vehicles, and by the different capacity groups in each class, a calculation was made based on certain assumptions as to annual mileage and rate of gasoline consumption In the case of passenger cars the assumptions were an average mileage of 7,000 miles per year and a consumption rate of 14 miles per gallon In the case of other vehicles an approximate relation between gross weight and gasoline consumption was used Privately owned trucks were assumed to travel 10,000 miles per year, and greater mileages were assumed for contract and common carrier trucks, tractor-trucks, taxicabs, and public carrier busses, because of the nature of their operations In the case of each State the total computed gallonage was adjusted to equal the net gallonage reported by the State for the year 1932

SUMMARY OF ESSENTIAL FACTS DISCLOSED BY SURVEY

The primary object of this survey was to determine approximately the contribution of the owners of motor vehicles to Federal, State, and local government The following tabulation gives the major items of motor-vehicle revenue and the total sums received in 1932, so far as it was possible to obtain the information

State fees and taxes		
Vehicle registration fees (motorcycles, trailers, and semitrailers included)	•$293,189,177	
Special fees, paid chiefly by for-hire vehicles	5,230,792	
Operators and chauffeurs licenses	18,280,802	
Miscellaneous motor-vehicle fees	9,475,924	
Gasoline or motor-fuel taxes	513,047,239	
Total		$839,223,934
County fees and taxes (incomplete)		1,703,000
Municipal fees and taxes (incomplete)		14,158,000
Personal property taxes, all jurisdictions (incomplete)		36,000,000
Federal excise taxes		92,404,000
Grand total		$983,488,934

This total of approximately a billion dollars is, as noted above, incomplete The State totals are believed to approach very closely to the true figures, although it is probable that in some States a few items escaped the attention of the investigators Certain of the reported payments, such as dealers' license fees, fines and penalties, and miscellaneous small items, which could not be regarded as direct imposts on the motor-vehicle owner, have been omitted Nominal fees charged against publicly-owned or official cars, have been included with the miscellaneous fees

The total number of vehicles (including motorcycles, trailers, and semitrailers, and excluding those publicly owned) registered in 1932, was 24,619,602 Dividing the total payment of $983,488,934 among these vehicles, we find that the average motor vehicle owner paid in 1932 a tax of $39 95 for the privilege of operating on the streets and highways of the nation

Only in the case of State taxes was it possible to separate the payments by different classes and sizes of vehicles Some of the salient facts along this line are brought out in the following paragraphs

Numbers of vehicles The following tabulation gives the total number of vehicles registered and tax-paid in the United States in 1932, grouped according to type of vehicle and class of service.

Passenger cars		
Private cars (including taxicabs in 20 States)	20,759,140	
Taxicabs (reported separately by 27 States and the District of Columbia)	77,222	
All passenger cars		20,836,362
Busses		
School (reported separately by 16 States)	9,813	
Contract, including sight-seeing (reported separately by 19 States and the District of Columbia)	3,314	

Public carrier (including other busses in States where not separately reported)	36,325	
All busses		49,452
Trucks and tractor trucks		
Privately owned and operated (including for-hire carriers where not reported separately)	3,144,704	
Contract carrier (reported separately by 26 States; includes common carriers in 3 States)	69,840	
Common carrier (reported separately by 31 States)	14,771	
All trucks and tractor trucks		3,229,315
Trailers and semitrailers		
Privately owned and operated (including for-hire carriers where not reported separately)	406,431	
Contract carrier (reported separately by 20 States; includes common carriers in 2 States)	5,008	
Common carrier (reported separately by 24 States)	3,837	
All trailers and semitrailers		415,276
Motorcycles		89,197
All vehicles (including trailers and motorcycles)		24,619,602

Analysis of the reported figures to determine the distribution of vehicles by rated capacity resulted in the following totals:

	Number	Percentage
Busses		
Up to 7 passengers	6,986	14 13
8 to 20 passengers	8,764	17 72
Over 20 passengers	27,122	54 84
Not classified by capacity	6,580	13 31
Total	49,452	100 00
Trucks and tractor trucks·		
1½ tons and less	2,681,985	83 05
Over 1½ tons and less than 3	341,285	10 57
3 tons and less than 5	128,938	3 99
5 tons	28,544	88
Over 5 tons	44,056	1 37
Not classified by capacity	4,507	14
Total	3,229,315	100 00
Trailers and semitrailers		
1½ tons and less	326,183	78 55
Over 1½ tons and less than 3	25,705	6 19
3 tons and less than 5	23,781	5 73
5 tons	6,372	1 53
Over 5 tons	8,714	2 10
Not classified by capacity	24,521	5 90
Total ..	415,276	100 00

Percentages The owners of passenger cars, which constituted 84 3 per cent of all vehicles, paid 72 9 per cent of the registration fees and 73 1 per cent of the gasoline taxes

Taxicabs, reported separately from passenger cars in 27 States and the District of Columbia, comprised 0 3 per cent of the vehicles, they contributed 0 5 per cent of the registration fees and 0 9 per cent of the gasoline taxes

Motor busses, which included 0 2 per cent of the vehicles, contributed 1 2 per cent of the registration-fee payments and 1 2 per cent of the gasoline-tax payments

TABLE I

AVERAGE PAYMENTS

Vehicle	Registration Fees	Gasoline Taxes	Special Fees	Total
Passenger Cars	$10 28	$18 07		$28 35
Taxicabs, reported by 27 States and District of Columbia	17 72	60 64	$1 53	79 89
Busses				
School, 16 States	21 57	39 17		60 74
Sightseeing and Other Contract Carriers, 19 States and District of Columbia	47 49	100 12	5 24	152 85
Public Carriers[1]	83 90	148 17	62 96	295 03
Trucks and Tractor Trucks				
Privately owned and operated	21 66	38 49		60 15
Contract Carriers, 23 States	31 14	62 44	14 77	108 35
Common Carriers, 30 States	32 93	78 10	72 46	183 49
Trailers and Semi-Trailers				
Privately owned and operated	7 71			7 71
Contract Carriers, 20 States	35 74		26 42	62 16
Common Carriers, 24 States	34 15		102 50	136 65

[1] Includes sight-seeing and contract busses in those States which made no segregation

Motor trucks and tractor trucks, comprising 13 1 per cent of the vehicles, paid 24 2 per cent of the registration fees and 24 7 per cent of the gasoline taxes.

Trailers and semitrailers constituted 1 7 per cent of the vehicles and paid 1.2 per cent of the registration fees

Motorcycles, constituting 0 4 per cent of the vehicles, contributed 0 1 per cent of the registration fees and 0 1 per cent of the gasoline taxes

Special fees to the amount of $5,230,792 were collected from operators of vehicles for hire and, in a few States, from certain classes of private operators These imposts took the form of mileage, ton-mile, or passenger-mile taxes, receipts taxes, special weight or capacity taxes, franchise fees or privilege taxes, fees for certificates of convenience and necessity, etc To this amount must be added $500,807 in excess registration fees paid by for-hire carriers of property (fees in excess of those calculated on the basis of the private carrier rate)

TABLE II

VARIATION OF FEES WITH CAPACITY

Capacity	Average Registration Fee	Average Motor-Fuel Tax	Average of all Fees[1]
Public Carrier Busses			
Seating Capacity			
7 or less	$25 84	$52 50	$104 77
8 to 20	57 39	123 13	234 50
Over 20	99 20	170 26	329 98
All Trucks and Tractor Trucks			
Rated Capacity in Tons	[2]		
1½ and less	$15 51	$36 16	$51 85
Over 1½ and less than 3	38 21	46 33	85 92
3 and less than 5	67 33	59 89	130 40
5	97 57	68 10	168 49
Over 5	113 89	81 12	199 60
All Trailers and Semi-Trailers			
Rated Capacity in Tons	[2]		
1½ and less	$3 34		$3 42
Over 1½ and less than 3	14 08		15 63
3 and less than 5	36 46		41 71
5	59 21		69 32
Over 5	70 55		92 35

[1] Includes Special Fees

[2] Includes excess registration fees paid by for hire carriers

If the special fees are added to the registration fees and gasoline taxes it is found that busses contributed 1 5 per cent and trucks and trailers 25 1 per cent of all fees and taxes directly imposed on motor vehicles in 1932

Average payments Average payments of registration fees and gasoline taxes by passenger cars were as shown in Table I

Variation of average payments with capacity As the weight or capacity of vehicles increases there is, in general, a steady rise in the average fees paid This fact is shown by the averages in Table II

DISCUSSION OF AVERAGE PAYMENTS BY DIFFERENT CLASSES OF VEHICLES

Passenger cars The rates of registration fee charged against passenger cars varied rather widely The lowest rate was found in the District of Columbia, where a one dollar fee is charged for the registration of all gasoline-driven vehicles In Vermont the rate varied from $16 for cars weighing 2,000 pounds or less to $43 for those weighing more than 4,500 pounds The average payment was $21 45, the highest in any State Other States which exacted relatively high fees from their passenger cars were Oregon, in which the average payment was $21 44, Arkansas, with $18 56, Connecticut, with $16 84, and Florida, with $16 25 Among the States in which the payments were low were Washington, with an average of $2 94, California, with $3 02, and Arizona, with $3 50 Clustered about the national average were Kansas, with $10 06, Pennsylvania, with $10 29, and Wyoming, with $10 32

Total payments by passenger cars were naturally highest in those States in which the gasoline-tax rate was 6 or 7 cents per gallon Thus in Florida, with a rate of 7 cents, the average payment of all fees and taxes by passenger cars was $59 90, in Arkansas, with a rate of 6 cents, the average payment was $50 38 In the District of Columbia, combining a $1 registration fee with a gasoline tax rate of 2 cents per gallon, the average of all fees and taxes was $10 77, the minimum for the country California came next, with $18 98 New York, with an average payment of $28 75, approached the national average of $28 35, as also did Connecticut, with $28 80, Arizona, with $28 96, and Texas, with $27·56

Busses As the numbers of privately owned school busses and of sight-seeing and contract-carrier busses which were reported in this survey form so small a portion of all busses reported, a discussion of the variation in their rates or payments is hardly significant, the following remarks are therefore confined to public-carrier busses

Disregarding the $1 fee in the District of Columbia, we find that the lowest average payment of registration fees by public-carrier busses, $11 24, was recorded in Montana In that State busses were classed as trucks and paid registration fees based on rated capacity, at 200 pounds per passenger, varying from $10 for 1 ton or less to $200 for more than 5 tons The highest average payment of registration fees, $517 59, was reported for public-carrier busses in Minnesota, where the fee was based on value (list price), the rate being 10 per cent for the first year and decreasing with succeeding registrations

Average payments of all fees and taxes, including special fees in 37 States and the District of Columbia, varied from $112 13 in Montana to $931.72 in Iowa. Other States in which the payments were high were Wisconsin, with $827 82, Florida, with $771 70, and Minnesota, with $637 90 The average for the country was $295 03

Space does not permit a detailed discussion of payments by busses in the various States To show the character of the variations which occur in tax rates and average payments, three States are selected as examples, Iowa, Montana, and Louisiana, these States being representative of maximum, minimum, and average payments

Public-carrier busses in Iowa paid registration fees on the basis of one per cent of value plus 40 cents per 100 pounds list weight and in addition a tax of one-fourth cent per gross ton-mile The State Motor Vehicle Department annually fixes the value of the various busses and the list weight The ton-mile tax is based on the passenger capacity multiplied by 150 pounds, plus the weight of the vehicle The gasoline-tax rate in Iowa is 3 cents a gallon

In Montana, in addition to registration fees paid on the basis described previously, public-carrier busses were required to pay a Public Service tax of $10 each The gasoline-tax rate is 5 cents per gallon

In Louisiana busses were required to pay registration fees of 68 cents per horsepower plus a fee based on seating capacity varying from $2 per seat for those seating 7 passengers or less to $7 per seat for those seating 30 passengers or more No special fees were charged The gasoline tax rate is 5 cents a gallon

These various rates yielded average payments as follows

Passenger capacity	Iowa	Montana	Louisiana
7 or less	$146 46	$78 48	$103 61
8 to 20	584 79	153 15	256 51
Over 20	1,211 62	(none reported)	458 63
All busses	931 72	112 13	295 10

Trucks and tractor trucks The vast majority of trucks and tractor trucks were either privately owned and operated or were reported without segregation by class of service Disregarding for the moment variation in payments by trucks of different sizes, we find average payments of registration fees varying from $9 40 in Nebraska to $50 68 in Vermont, with a national average of $21 66 In payments of all fees and taxes (registration fees and gasoline taxes) the averages vary from $32 50 in Missouri to $114 38 in Florida, the figure for the country as a whole being $60 15

To illustrate the range in average payments by trucks of different rated capacities, the figures for Georgia, Missouri, and New York are given in Table III These three States are fairly typical of the range in average payments of registration fees and gasoline taxes

In Georgia the rates of registration fee varied from $15 for trucks of 1 ton capacity or less to $375 for 6-ton trucks The rate on 7-ton trucks was $750, and that on trucks of more than 7 tons was $1,125, but there were no trucks of more than 6 tons capacity registered in

Georgia in 1932, and only one 6-ton truck The gasoline-tax rate in Georgia was 6 cents per gallon

In Missouri the rates varied from $10 50 for trucks of 2 tons capacity or less to $36 for 7-ton trucks, with $15 added for each additional ton The gasoline-tax rate was 2 cents per gallon

TABLE III

COMPARISON OF AVERAGE PAYMENTS BY TRUCKS OF DIFFERENT RATED CAPACITIES REGISTERED IN GEORGIA, MISSOURI, AND NEW YORK IN 1932, INCLUDES PRIVATE TRUCKS ONLY IN GEORGIA AND MISSOURI, ALL TRUCKS IN NEW YORK

Rated capacity in tons	Number registered and tax-paid	Percentage distribution	Average per vehicle payments		Registration fees and gasoline taxes		
			Registration fees	Gasoline taxes	Total payments	Percentage distribution	Average per vehicle
Georgia							
1½ and less	38,973	95 51	$16 37	$68 93	$3,324,544	93 41	$85 30
Over 1½ and less than 3	1,572	3 85	28 65	93 15	191,472	5 38	121 80
3 and less than 5	256	63	43 46	117 45	41,195	1 16	160 91
5	4	01	133 93	143 38	1,110	03	277 31
Over 5	1	00	375 00	175 74	551	02	550 74
Total or average	40,806	100 00	17 04	70 17	3,558,872	100 00	87 21
Missouri							
1½ and less	88,009	89 67	$9 51	$21 11	$2,694,389	84 45	$30 62
Over 1½ and less than 3	5,992	6 10	16 44	28 53	269,480	8 45	44 97
3 and less than 5	3,791	3 86	17 04	35 97	200,955	6 30	53 01
5	333	34	26 19	43 91	23,343	73	70 10
Over 5	26	03	34 73	53 83	2,302	07	88 56
Total or average	98,151	100 00	10 28	22 22	3,190,469	100 00	32 50
New York							
1½ and less	216,396	69 28	$20 77	$29 51	$10,879,905	52 37	$50 28
Over 1½ and less than 3	64,326	20 59	41 92	39 88	5,261,508	25 32	81 80
3 and less than 5	14,012	4 49	69 17	50 28	1,673,792	8 05	119 45
5	6,695	2 14	84 03	61 38	973,524	4 69	145 41
Over 5	10,945	3 50	106 38	75 24	1,987,787	9 57	181 62
Total or average	312,374	100 00	31 65	34 86	20,776,516	100 00	66 51

New York imposed on trucks weighing less than 1,800 pounds a flat rate of $12 and on trucks weighing 1,800 pounds or more a fee of 80 cents per hundredweight on the unladen weight of the vehicle The gasoline-tax rate was 2 cents

Georgia is an example of a State in which there is a wide range from relatively low fees for light trucks to very high fees for heavy trucks

In Missouri the rates were uniformly low, while in New York, with fees based on net weight, the average payments for all sizes of trucks were not far from the national averages The high gasoline-tax rate in Georgia brought the average payments of all fees and taxes well above those in Missouri and New York for all capacity groups

An interesting point to be observed in Table III is the relative distribution of trucks of different sizes in the three States In Georgia nearly 96 per cent of trucks were of $1\frac{1}{2}$ tons capacity or less, while in New York less than 70 per cent were in this capacity group With only five trucks of 5 tons capacity or more, Georgia obtained a negligible revenue from heavy trucks New York, on the other hand, derived nearly three million dollars from these two classes It should be recognized that we are comparing here two States which are very different in density of population and in industrial and commercial development It appears probable, however, that the imposition of fees running as high as $1,125 has discouraged the registration of heavy trucks in Georgia This inference is supported by data obtained from other States in which the registration rates were high

In Alabama the rate of registration fees on 5-ton trucks was $400, on 6-ton trucks, $750, and on 7 tons and over, $1,000 Only seven 5-ton trucks were reported, one 6-ton, and none over 6 tons The total registration fees paid by these eight trucks were $2,487

In Arkansas the law provided registration fees of $300 for 5-ton trucks and $400 for 6-ton trucks, those of more than 6 tons capacity not being permitted For-hire carriers paid 50 per cent in excess of the private rate Only three trucks of 5 tons capacity or more were registered in this State in 1932, paying total registration fees of $850.

In North Dakota, where the rate on 5-ton trucks was $400 and on those over 5 tons, $600 to $1,500, with reductions after the first year of registration, we find six 5-ton trucks paying a total of $1,404, and none over 5 tons In South Dakota, where the schedule of rates, based on chassis weights, was also very high, only four 5-ton trucks were registered, with total collections of $1,300

In Louisiana the law provided that trucks with a net carrying capacity greater than 5 tons should be charged $150 for each thousand pounds carrying capacity in excess of 5 tons, in addition to a fee of 68 cents per horsepower and $32 per ton for the first 5 tons ($50 per ton for common carriers). City trucks were exempted from this schedule and paid at the rate of $10 per ton plus 68 cents per horsepower No private trucks were reported in 1932 as paying the high rates cited above. Five common carriers having capacities of more than 5 tons paid a total of $323 Of city trucks there were 28 of more than 5 tons capacity and the total collections from them were $3,050 Thus we have a relatively low rate of registration fee producing nearly ten times the revenue obtained from a very high rate applied to vehicles of the same capacity group

For-hire carriers As an example of the variation in fees of private, contract, and common carrier trucks it is interesting to compare Iowa, Missouri and North Carolina In Iowa the registration fee was based on capacity for all classes and varied from $15 00 for a ½-ton truck to $450 for a 9-ton truck Contract-carrier trucks were charged an annual franchise fee of $5 00 in addition, while common carrier trucks paid a ton-mile tax of one-fourth cent per ton-mile

In Missouri contract trucks were assessed the same rates for registration as private trucks, based on carrying capacity, and paid no additional fees Common carrier trucks, in addition to the capacity registration fee, paid a special load capacity tax ranging from $25 for a truck of 1½ tons to $500 for a truck over 9 tons

TABLE IV

AVERAGE PAYMENTS BY TRUCKS PRIVATELY OWNED AND OPERATED, CONTRACT CARRIERS, AND COMMON CARRIERS IN IOWA, NORTH CAROLINA, AND MISSOURI IN 1932

State and item	Privately owned and operated	Contract carrier	Common carrier
Iowa			
Registration fees	$20 17	$52 83	$73 64
Gasoline tax	23 22	43 44	46 99
Franchise fees	—	5 00	—
Ton-mile tax	—	—	131 39
All fees	43 39	101 27	252 02
Missouri			
Registration fees	10 28	15 26	13 56
Gasoline tax	22 22	40 24	37 51
Special capacity tax	—	—	48 38
All fees	32 50	55 50	99 45
North Carolina			
Registration fees	20 25	63 65	42 04
Gasoline tax	65 01	113 65	112 47
Special capacity tax	—	—	42 91
All fees	85 26	177 30	197 42

In North Carolina all trucks were required to pay registration fees based on gross weight On private trucks the rate varied with capacity from 55 cents to one dollar per hundredweight, on contract carriers, from one to four dollars, on common carriers, 90 cents per hundredweight regardless of capacity Common carrier trucks were subject to an additional tax of 6 per cent of gross receipts if and when and in such amounts as this exceeded the original registration payment

The gasoline-tax rates in Iowa, Missouri, and North Carolina were 3, 2, and 5 cents per gallon, respectively

The average payments by private, contract, and common carrier trucks in these three States are shown in Table IV It will be noted that in Iowa, while the fees paid by trucks privately owned and operated were moderate, the average payments by common carriers were unusually high In North Carolina, largely because of the five-cent tax on gasoline, the payments by private trucks were large, and those of contract and common carriers were progressively higher In Missouri payments by all classes were relatively low

Trailers and semitrailers No unusual features were found in the taxation applied to trailers and semitrailers Light trailers were usually charged little more than nominal fees, the average registration fee paid by trailers and semitrailers, privately owned and operated, of 1½ tons capacity or less, was $3 29. Much higher fees were charged against heavy trailers, the average for those of more than 5 tons capacity being

TABLE V

COMPARISON OF PERCENTAGE OF ALL VEHICLES REGISTERED WITH PERCENTAGE OF REGISTRATION FEES AND GASOLINE TAXES PAID, FOR PASSENGER CARS AND TAXICABS, BUSSES, TRUCKS AND TRACTOR TRUCKS, AND TRAILERS AND SEMITRAILERS, IN MISSOURI, NEW YORK, AND ARIZONA

Type of vehicle	Missouri		New York		Arizona	
	Percentage of all vehicles	Percentage of all registration fees and gasoline taxes paid	Percentage of all vehicles	Percentage of all registration fees and gasoline taxes paid	Percentage of all vehicles	Percentage of all registration fees and gasoline taxes paid
Passenger cars and taxicabs	84 95	81 71	85 26	71 75	82 63	66 76
Busses	03	11	27	1 48	28	2 07
Trucks and tractor trucks	13 64	18 00	13 86	26 52	15 17	30 47
Trailers and semitrailers	1 38	18	61	25	1 92	70
All vehicles	100 00	100 00	100 00	100 00	100 00	100 00

$73 91 In a few States the rates on large trailers were very high, thus we find in South Dakota a rate of $500 applied to trailers weighing more than 5,000 pounds of which only one was registered in 1932

In general those States which imposed special fees on for-hire trucks and tractor trucks required similar payments for the operation of trailers and semitrailers

Comparison of percentage payments by different classes of vehicles The variation in the relative distribution of registration fees and gasoline taxes among the several types of vehicles in the different States is illustrated in Table V, which shows the percentage of total vehicles registered in each class and the corresponding percentage of total registration fees and gasoline taxes paid by each class for the States of Missouri, New York, and Arizona Payments by motor cycles are not included in this tabulation

Missouri shows an unusually even distribution of taxes among the several types of vehicles in proportion to their numbers, while Arizona exemplifies a very uneven distribution In Missouri, passenger cars and taxicabs, representing 85 per cent of the vehicles registered, paid 82 per cent of the total registration fees and gasoline taxes, while in Arizona where passenger cars and taxicabs were nearly as large a proportion of vehicles registered as in Missouri (83 per cent), they paid only 67 per cent of the taxes Trucks in Missouri were about 14 per cent of all vehicles, but paid 18 per cent of the taxes, while in Arizona the corresponding percentages are 15 and 30 New York represents a distribution of vehicles and taxation which is fairly typical, passenger cars and taxicabs comprising 85 per cent of the vehicles and paying 72 per cent of the taxes, while trucks were 14 per cent of the vehicles and paid 27 per cent of the total registration fees and gasoline taxes It will be noted that these comparisons are made for registration fees and gasoline taxes only because these are the only taxes which are paid by all classes of vehicles (trailers excluded) and thus establish a uniform basis of comparison

LOCAL AND PERSONAL PROPERTY TAXATION

Taxation of motor vehicles by local authorities—counties, municipalities, and lesser jurisdictions—has been an almost unexplored field The intricacies in forms and methods of taxation presented by 48 States and the District of Columbia are magnified manyfold when the inquiry is carried down into the subordinate taxing jurisdictions, numbering many thousands The present investigation, therefore, may be regarded as the first serious attempt to learn something about the local phases of motor vehicle taxation, and its results, while not entirely complete, are indicative of considerable amounts paid into local general tax funds by motor vehicle owners

The collections in 1932, as reported through answers to questionnaires, may be summarized as follows, the figures being rounded off to the nearest thousand

County taxes on motor vehicles	$1,703,000
Municipal taxes on motor vehicles	14,158,000
Personal property taxes on motor vehicles (State, county, and other jurisdictions)	36,000,000
Total	$51,861,000

It would appear that in only five States did the counties impose taxes (other than personal property taxes) upon motor vehicles in 1932, and that four of these States are located in the South and are contiguous to each other The five States are Alabama, Louisiana, Mississippi, Nevada, and Tennessee Gasoline taxes collected by the counties of Louisiana, Alabama, and Mississippi accounted for about 90 per cent of all the special county taxes reported throughout the United States The largest single sum reported in any State was that for county (parish) gasoline taxes in Louisiana, which totaled approximately $1,139,000

Municipal fees and taxes on motor vehicles are much more common than county fees and taxes, being found in 39 States The most prevalent type of fee, although not the most productive, was found to be a franchise tax, frequently levied upon taxicabs, busses, and for-hire trucks, and sometimes upon occupations in which motor vehicles are used, such as drayage and the like Fees of this nature were reported in 34 States, and the total collected was $2,053,000

Municipal gasoline taxes were found in only six States, in widely separated regions of the South and West, but the total collected was $2,602,000, of which Missouri alone accounted for approximately 67 per cent

Registration fees amounted to $9,010,000, and of this total the State of Illinois alone accounted for approximately 68 per cent

Other fees reported included chiefly such items as chauffeurs' and operators' licenses, to the total amount of $402,000

Among the 39 States reporting special municipal fees, the States of Illinois and Missouri far exceeded any others and together represented about 67 per cent of the National total

Some form of taxation of motor vehicles as personal property was found to exist in 31 States and the District of Columbia This phase of the tax problem is greatly complicated by the large number and variety of taxing jurisdictions which may place imposts upon personal property, among which are the State, the county, the municipality, and the many lesser jurisdictions, such as school districts, road districts, sanitary districts, and the like The total personal property tax levied upon motor vehicles in 1932 was approximately $44,000,000, of which about $36,000,000 was actually collected The survey disclosed that 17 States, in which there were 11,834,000 motor vehicles registered, or 46 per cent of the total for the Nation, exempted motor vehicles from taxation as personal property

FEDERAL TAXATION

A new element entered the motor vehicle taxation field in 1932 in the form of the United States Government itself, with manufacturers' excise taxes levied upon gasoline, lubricating oils, tires and inner tubes, automobiles and motorcycles, automobile parts and accessories, and trucks Administration of these new taxes began in June 1932, and the total collected during the second half of the year was $84,293,846—this sum representing actual collections, which were considerably less than the tax-earnings during that period, due to lag in collections The true earnings of these taxes in 1932 are shown by the collections of the period ending January 31, 1933, which were $98,161,386 After adjustment for other than motor-vehicle uses of gasoline and oil, it appears that the total payments attributable to motor vehicles were approximately $92,404,000

REPORT OF COMMITTEE ON TRANSPORTATION ECONOMICS

R L MORRISON, *Chairman*

Professor of Highway Engineering and Highway Transport, University of Michigan

THE ENGINEERING VALUATION OF HIGHWAY SYSTEMS

A PROBLEM IN PUBLIC UTILITIES VALUATION

BY ANSON MARSTON

Senior Dean of Engineering, Iowa State College

SYNOPSIS

Highway engineering science is still crude and imperfect in two important branches first, engineering valuation of highways, second, highway accountancy The highway systems constitute a public utility comparable to the railways and surpassing all other utilities in magnitude of investment, yet highway engineers do not know how to ascertain the correct present values of the highway properties nor have they developed any standard system of accountancy

The same general principles of valuation and accountancy developed for other public utilities apply to highway valuation including the rule that all factors affecting value must be given due consideration and such weights as may be just and right, as determined by judgment, not by formula

The physical value of the system may be determined by (1) valuing the land owned and used, (2) determining the original cost new less depreciation, (3) determining reproduction cost less depreciation, giving such weight as is just to original cost and reproduction cost price levels Allowance for preliminary expense and going concern intangible values are matters for research and discussion as is also the working capital of highway systems

The earning values of highway systems can be determined by analyzing the revenues (from property taxes, license fees and fuel taxes), operation costs and annual "actual" depreciation costs

The service-worth values are what their earning values would be if their revenues were just equal to the "reasonable worths" of their total highway services rendered Traffic surveys and research on reasonable worths of different highway services are necessary to determine true service worth values

A standard accountancy system should provide for current depreciation accountancy and include property ledger sheets kept up to date Depreciation of the various property units and / or age groups of like units should be estimated by modern methods

INTRODUCTORY DISCUSSION

Immediately after the World War, the author had the great privilege of assisting actively in the formation of the Highway Research Board

There were two principal reasons for its creation: First, it was clearly evident that many billions of dollars were about to be spent in constructing improved highways in the United States, second, in the face of this enormous impending expenditure, the science of modern highway design, construction and maintenance was so pitifully crude and imperfect that enormous wastes were bound to occur, resulting from sheer ignorance, unless unprecedentedly great, immediate scientific advances could be accomplished by a coordinated national program of highway research It is now most gratifying to know that the Highway Research Board has been wonderfully successful in placing modern highway engineering upon a truly scientific basis The value of the highways we have constructed since 1919 is hundreds of millions of dollars greater than could have been obtained without the scientific highway engineering discoveries made the last 14 years

Nevertheless, present day highway engineering science is still pitifully crude and imperfect in two very important branches First, the engineering valuation of highways, second, highway accountancy

During the last 14 years, the highway engineers of the United States have spent about ten billions[1] of dollars for highway construction alone, yet we do not know, and we have developed no way to ascertain correctly the true present values of the highways which have been built at such great cost

The highways of the United States constitute one of the greatest public utilities in the world, whose cost is more than half that of all the railways in our country, yet we have developed no property ledger system worthy the name for accounting for this tremendously valuable property trusted to the care of highway engineers

The annual highway revenues collected in the United States in the form of road taxes on property, gasoline taxes and vehicle license fees constitute one of the major parts of our crushing tax burdens, yet we have never made the researches necessary to establish a scientific basis for equitable charges for highway services and their equitable division between property taxes, fuel taxes and personal fees, all of which are still determined mainly by legislative whims instead of by just principles

Furthermore, highway engineers have never even developed, much less put into use, any standard system of highway accountancy, the first essential of wise public utility management and regulation Ignorant of recent progress in the scientific treatment of depreciation, present day highway engineers content themselves with wild guesses at annual highway depreciation costs, though they are a very important part of the true costs of highway services

A large amount of carefully planned and executed highway research is still needed to develop the scientific technique necessary for the

[1] Omitting maintenance costs and all but a small percentage of the costs of constructing city streets

correct valuation of highway systems, and for a comprehensive and satisfactory standard system for highway accountancy

HIGHWAY VALUATION A SPECIAL CASE OF PUBLIC UTILITIES VALUATION

It is necessary to realize that our highway systems are great public utilities, of which highway engineers and other officials are the responsible managers, not merely the promoters and builders Already the total investment in the highways of the United States is more than half her railway investment, and surpasses the national investment in nearly every other utility The general principles of valuation and accountancy which have been developed for other public utilities during 50 years of experience, study and litigation apply also to highway valuation and accountancy The fact that highway systems are owned by public corporations (states, counties, townships) instead of by private stock companies does not change their status as public utilities, "endued with a public character"

However, highway system public utilities have certain distinctive characteristics, some of which are The capital invested in highways is partly obtained on public credit which is not dependent on the value of highway property, the different state, county and township highway systems connect and exchange traffic at multitudinous points, there are as yet only inadequate traffic survey records of the amount of highway services rendered by the different highway systems, the charges for highway services are not collected directly, in accordance with scheduled rates for different kinds of service, but indirectly, in the form of property taxes, license fees and gasoline taxes, whose proceeds are distributed arbitrarily between the different systems

GENERAL PUBLIC UTILITY VALUATION PRINCIPLES APPLICABLE TO HIGHWAY SYSTEMS VALUATION

The same *general principles of valuation and accountancy* which have been developed for other public utilities apply also to highways *All* factors affecting value must be duly considered and be given "such weight as is just and right in each case" For valuation, the highways of each state can be divided into systems, such as the one state and the ninety-nine county highway systems of Iowa Each system can be valued separately The first step in valuation should be to prepare a complete inventory, showing all property units

Physical Value The physical value of the property may be determined by (1), valuing the land owned and used, (2) and (3), determining separately the original and the reproduction costs new, including overhead costs, less depreciation, of all other physical property units, (4), giving such weight as is just and right in each case to original cost and to reproduction cost price levels

Intangible Values Whether any, and if any what, allowances should be made for preliminary expense value, going concern value and for the value of highway easements over lands used but not owned are all still matters for research and discussion

Working Capital The working capital of a highway system is the average amount of *operation* funds kept on hand between their dates of receipt and expenditure

Earning Value The earning value of a highway system is the capitalized value of the actual average annual existing net returns For any one year, *net return* = *total revenue* (say from property taxes, license fees and gasoline taxes)—*operation costs* (Omitting owners' vehicle operation costs)—*annual "actual" depreciation* Interest is a part of net return, *not* an operation cost

Service-Worth Value The service-worth value of a highway system is what its earning value would be if its total annual revenue were just equal to the total "reasonable worth" of the total annual highway services rendered A large amount of work on traffic surveys and in researches on the reasonable worths of different classes of highway services is necessary before the true service-worth values of particular highway systems can be determined

Standard Highway Accountancy System The American Association of State Highway Officials and other highway organizations ought co-operatively to develop a standard system of highway accountancy, including current depreciation accountancy, and standard property ledger sheet accounts, kept constantly up to date

Determination of "Actual" Depreciation "Actual" depreciation costs are a very important part of the costs of highway services and should be estimated separately for each particular property unit, or age-group of like units, by the modern methods recently developed These modern methods are based on the use of mortality curves in estimating probable service lives, on probable service lives reestimated from time to time during actual service, and on "probable future operation return ratios" which take due account of changes in the annual values of the services rendered by particular units The depreciation determinations are kept adjusted to conform with the reestimated ratios and probable service lives, so that they always check out correct at the actual dates of the retirements of the different units

Doubtless, the most important general valuation principle applicable to highway valuation is the famous "*Smyth v Ames Rule,*" that *every* factor affecting value must be given due consideration and "such weight as may be just and right" in each case, as determined by sound judgment, *not* by any "formula" This rule has been upheld by the U S Supreme Court ever since 1898, in an unbroken line of decisions

In these decisions, the U S Supreme Court has consistently rejected both the "*Reproduction-Cost-New-Less-Depreciation*" valuation formula, which would give reproduction cost price levels dominant weight, and the "*Prudent Investment*" valuation formula, which would give original cost price levels dominant weight

The fundamental basis of exchange value is the "present worth" of *probable* future net returns These probable future net returns are indicated by sound judgment *at the date of valuation*, and are not likely to prove to be identical with the *actual* net returns which the future will bring

All the factors which affect value do so by affecting the *probable* future net returns. The factors which have been either actually enumerated or necessarily implied by the U S Supreme Court are:

1 The *original costs* new, including overhead costs, less depreciation to date, of the *existing* property units

2 The *reproduction costs* new, including overhead costs, less depreciation to date, of the *existing* property units

3 The *earning value* obtained by capitalizing the average present annual net returns.

4 The *service-worth value* obtained by capitalizing the average annual net returns which would be earned if the revenues collected were based on charges for highway services just equal to their "reasonable worths "

5. The *stock-and-bond value*, based on current market prices However highway systems have no stock-and-bond values, for they issue no stocks, and their outstanding bonds, if any, are based on the credit of the public corporations which issued them, not on the values of the highway properties

6 All "*other pertinent factors*," affecting value Such as, for highway systems, the characters of the lands and the communities served (which will affect future traffic), present and probable future business conditions, present and probable future price trends

IOWA HIGHWAY SYSTEMS

For valuation purposes, the highways of each state may well be divided into separate systems, each of which includes all highways owned and administered by a separate public corporation, such as a state, a county, a township For Iowa, these systems and the sums spent for their construction during the years 1919–1932, inclusive, alone, are as follows.

Iowa Highway Systems	Construction Costs during 1919–32, inclusive, alone
1 State System, Primary Roads	$232,542,000
99 County Systems, Secondary and Local Roads	120,414,000
	$352,956,000

No construction costs prior to 1919 are included in the above table, and new construction is still proceeding and will continue a long time, About $8,000,000 on the state system and perhaps half as much on the county systems are being expended on new construction this year The assessed valuation of Iowa railways this year is $215,000,000, including rolling stock In valuing the highways a large deduction would be made for depreciation, but even so, and without including the $300,000,000 or more invested in the automobiles and trucks which constitute the main part of highway "rolling stock," it is evident that Iowa highways constitute a public utility which already approximates her railways and far surpasses any other of her utilities in magnitude of investment

The records of construction costs, operation expenditures and revenues (from highway property taxes, license fees and gasoline taxes) are separate and distinct for each of Iowa's 100 highway systems, of each of which, separately, it is therefore feasible to determine the original cost value, the reproduction cost value and the earning value

On the other hand, a large amount of work on traffic surveys and a large amount of research upon the "reasonable-worths" of different kinds of highway services are necessary before the true "service-worth" values of any of the systems can be determined

THE PROCESS OF MAKING AN ENGINEERING VALUATION OF A HIGHWAY SYSTEM

The process of making an engineering valuation of any one of Iowa's highway systems might well be about as follows

1 Make a complete inventory of all highway property units and/or age groups of like units

There should be in every state, but at present are not in any state, complete highway property records, kept constantly up to date, on standard property ledger sheets, from which a classified inventory for each highway system condensed into valuation groups, can readily be prepared without extensive field work

Physical Value

2 Determine the value of the *land owned and used* This will be a small part of the land *used*, over most of which the public owns only a highway *easement* By the "law of the land," as repeatedly laid down by the U S Supreme Court, the value of the lands owned and used is equal to the *present market value of similar adjacent lands*

3 Determine the *original costs new, including overhead construction costs, less total actual depreciation to date*, of all *present existing* physical property units except land

In doing this, quite a number of omissions in the early records must be supplied by the valuator from his knowledge of cost levels prevailing in past years

4 Determine the *reproduction cost new, including overhead construction costs, less total actual depreciation to date*, of all *present existing* physical property units except land

In doing this the valuator may use "spot" reproduction cost prices, the average for the current year, or the "period" reproduction cost prices which he forecasts for a period of 3 to 5 future years[1]

5 Determine the *total present physical value* of the property of the system, by giving original cost and reproduction cost prices level the relative weights which are judged to be "just and right" in this case, in view of present and forecasted future price trends

Intangible Values

6 Determine whether any, and if any what, *intangible values* should be allowed for (1), *preliminary expense value;* (2), *going concern value,* (3), *highway easements, over lands used but not owned*

In general the courts allow the above intangible values for other public utilities, but research and study are needed to determine what should be the practice in valuing highway systems, owned and operated by the public, especially in cases where the revenues collected are insufficient to pay a "fair" net return profit above the actual costs of the highway services rendered

Working Capital

7 Determine the *working capital* required for the operation of the highway system It is equal to the *average* amount of *operation* funds *necessary* to keep on hand to pay operation cost expenditures promptly

Earning Value

8 Determine the *earning value,* if any, of the highway system The earning value is the capitalized value of the actual, present, average annual net returns The equation for calculating the net return for any one year is

Net return = *total highway revenue* − *total operation costs* (interest is *not* an operation cost) − *total annual "actual"* depreciation

Interest on capital indebtedness (such as bond interest) is a part of net return Owners' vehicle operation costs must be omitted.

In general, public utility earnings are subject to regulation, down or up, until they are just sufficient to yield a "fair net return" on the "fair value" of the property owned and used in rendering the services, *provided*[2] that this fair net return does not require charges for the services greater than their "reasonable worths"

Hence, to give material weight in valuation to the "fair net return

[1] In the past the average prices of the last 3 to 5 years have often been used, but should not unless they are believed to forecast the future correctly

[2] In the Smyth v Ames decision the U S Supreme Court said "What the company is entitled to ask is a fair return upon the value of that which it employs for the public convenience On the other hand, what the public is entitled to demand is that no more be exacted from it for the use of a public highway than the services rendered by it are reasonably worth"

on fair value" of public utility properties would throw the valuator into a "vicious circle of reasoning"

Highway systems are public utilities, owned and operated by the public for the purpose of affording adequate and satisfactory highway facilities for vehicles owned and operated by citizens, not by the utility The question of how large, if any, excess of highway revenues should be collected above the actual operation cost plus depreciation costs is as yet unsettled, and is worthy of extensive research, study and discussion

Service-Worth Value

9 Determine the *service-worth value* of the highway system, as nearly as may be The service-worth[2] value is what the earning value of the system would be if its total annual revenue were just equal to the total "reasonable worths"[2] of the total annual highway services rendered

In order to determine the service-worth value of a highway system, it is necessary

First, to conduct *traffic surveys*, in order to determine, as nearly as may be, the annual amounts of highway services of different classes rendered by the different component roads of highway systems

The traffic surveys may demonstrate that some unwisely planned, individual roads do not now and are not likely in the future to render highway services commensurate with their costs

Second, to determine the "reasonable worth,"[2] per unit, of each different class of highway services

In some few cases (such as present street railways) the "reasonable worths" of public utility services may be fixed by competition, but in the majority of cases, including most present highway systems, their "reasonable worths" are the costs at which the users could supply themselves with the same services, as organized corporations, not as individuals[3] The application of this principle to highway systems would make the total "reasonable worths" of their annual highway services just equal to the sum of their annual *operation costs* (omitting owners' vehicle operation costs) plus their annual *"actual" depreciation cost* plus their annual *"fair" net returns*, after eliminating any roads which the traffic surveys show to have been unwisely planned

The distribution of the total "reasonable worths" of the system's annual highway services between the different classes of such services requires a large amount of study and research not yet made This is an important subject for new highway research, and for much study and discussion. The following discussion is to be considered as tentative and suggestive only, not as stating final conclusions by the author

A tentative classification of highway services might be

(1) *Road access* highway service This service is direct to the *land*

[3] See *Brunswick & Topsham Water District v Maine* Maine Supreme Court 99 Me 371, 59 Atl 537, December, 1904

owners of adjacent properties; and justifies, in principle, the customary road taxes on lands All classes of roads, primary, secondary, local and by-, render this land access service

It is tentatively suggested that the total "reasonable worths" of the annual road access highway services of the highways within a region are the annual yields of road taxes on lands just sufficiently high to pay the major part of the total annual *operation costs* (omitting owners' vehicle operation costs) plus total *actual depreciation costs* plus annual *fair net returns* of a system of local roads coextensive with the actual highways, and of the type found to be most practicable in view of the topography, of the available road materials, and of the character and values of the lands and communities served

(2) *Road use* highway service This service is direct to *road users*, and justifies, in principle, the customary vehicle license fee and gasoline tax charges for road use The vehicle license fee is a form of "ready to serve charge", while the gasoline tax is probably the best approach practicable at present to ton-mile and vehicle-mile road use taxes

It is tentatively suggested that the total "reasonable worths" of the annual road use services rendered by a highway system is that portion of the sum of its annual *operation costs* (omitting owners' vehicle operation costs) plus its annual *actual depreciation costs* plus its annual *fair net returns* left after deducting the "reasonable worth" of its road access highway services, and that this total may be equitably divided between road services on by-roads, local roads, secondary roads and primary roads by suitably taking into account the average actual vehicle-mile and ton-mile costs of each of those road types

A STANDARD HIGHWAY ACCOUNTANCY SYSTEM

The American Association of State Highway Officials, the Highway Research Board and other highway organizations ought cooperatively to develop a standard system of highway accountancy

Such standard systems of accounts are already prescribed by the Interstate Commerce Commission for steam railways, electric railways and telephone companies, and have been developed and are recommended by the National Association of Railway and Utilities Commissioners for electric, water and gas utilities

In accordance with the rulings of the U S Supreme Court that actual depreciation *is* a real production expense, which must be provided for out of annual income before there is any real net return, and in accordance with the opinions now held and so often expressed by so many industrial authorities, the standard system of highway accountancy ought to provide fully for *current depreciation accountancy*, including a *depreciation reserve account* equal at all times to the total accrued "actual" depreciations on all property units, and for complete property ledger accountancy for each unit and/or age-group of like units

It is necessary for privately owned public utilities to make an annual depreciation appropriation out of income, equal to the total "actual" depreciation during the year In highway administration, all that is needed is to make sure that an investment equal to the year's depreciation is made, out of the current income of each year, in highway replacements, highway improvements and/or new highways

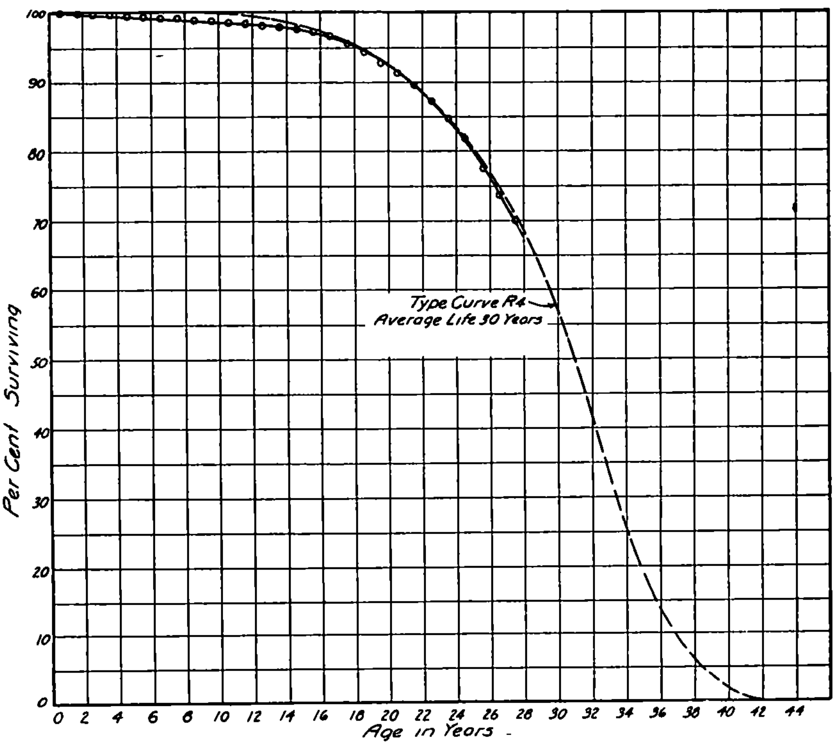


Figure 1 Survivor Curve of Alternating Current Motors by Annual Rate Method

THE MODERN METHODS OF DETERMINING "ACTUAL" DEPRECIATION

It seems not yet to be generally known that during the last ten years entirely practicable modern methods for determining "actual" depreciation have been developed which·

(1) Use mortality curves of different classes of property in estimating probable service lives

(2) By reestimating the probable service life of each property unit and/or age-groups of like units from time to time during service, and immediately afterwards adjusting depreciations accordingly, substitute actual observed service life for guessed average life and make the depreciation determination check out exactly with the value new less actually realized salvage value at the date of the actual retirement of each property unit.

(3) By the use of estimated and reestimated "probable future operation return ratios," enable correct depreciation allowances to be made for changes in the annual values of the services of property units

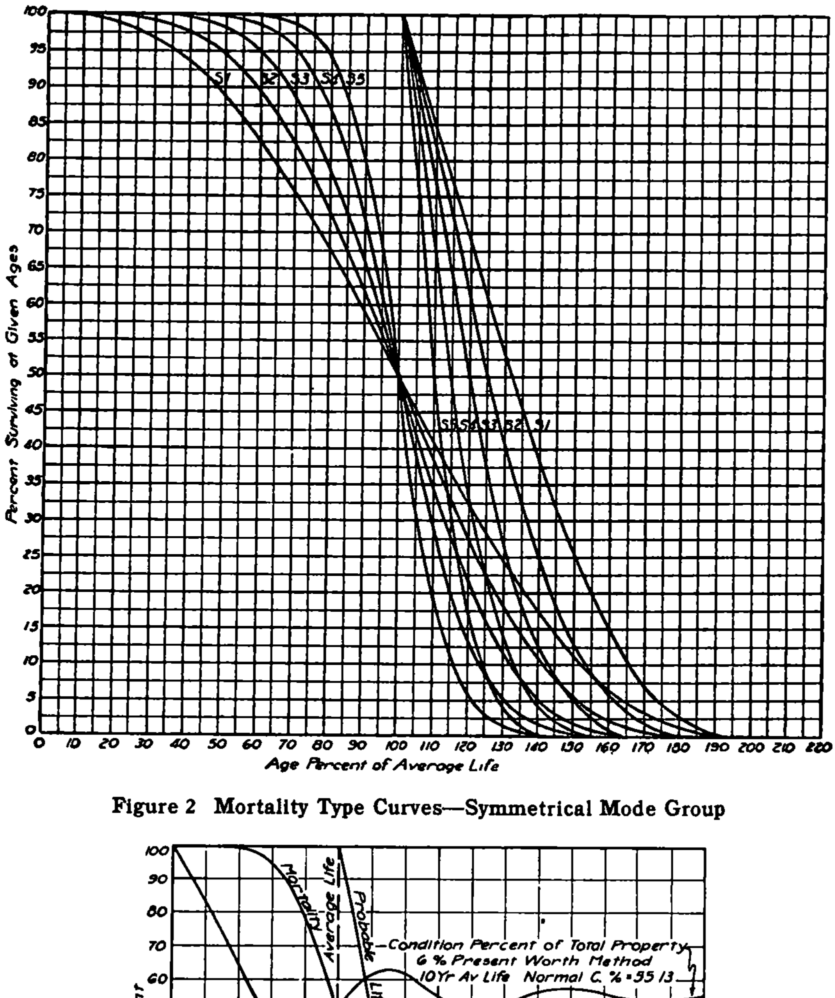


Figure 2 Mortality Type Curves—Symmetrical Mode Group

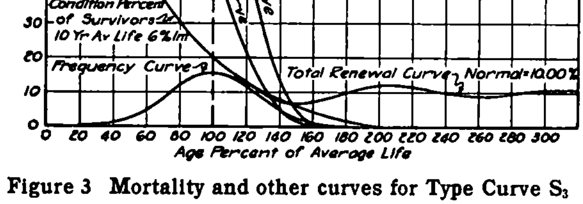


Figure 3 Mortality and other curves for Type Curve S_3

(4) By the use of extensive new "condition per cent" tables make it as easy to base depreciation determinations on the correct "present

worth" of probable future net returns basis of value as on the incorrect "straight line" assumption, that a dollar received as net return at a distant future date is worth today just as much as a dollar on hand today in cash

A very large part of the extensive researches by which these modern depreciation determination methods have been developed has been made during the last 14 years, at the Iowa Engineering Experiment Station, whose Bulletin 103, Life Characteristics of Physical Property, will be sent without charge to all who make request

Figure 1, herewith, shows an actual example of a mortality curve for alternating current electric meters, as determined from eight years records of retirements in a large electric light and power property

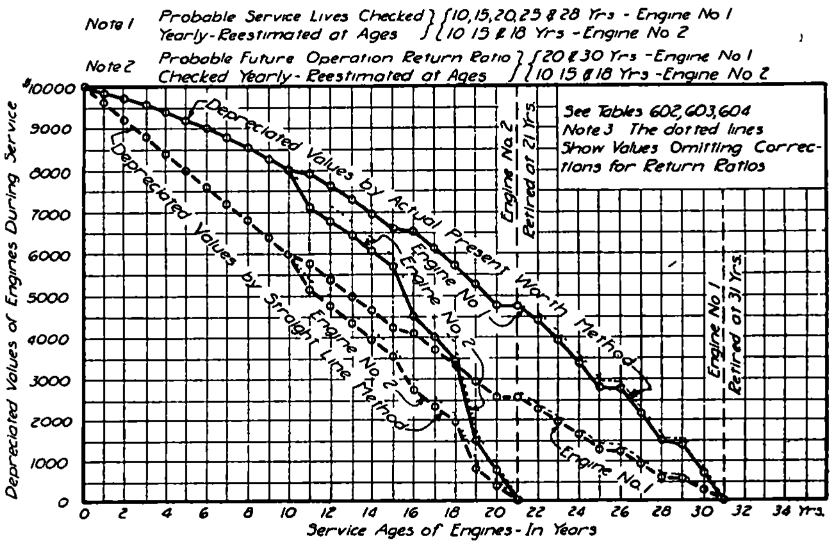


Figure 4 Depreciated Values of Two Duplicate Pumping Engines During Service Showing Actual Depreciations By Actual Present Worth Actual Depreciation Method By Straight-Line Actual Depreciation Method

The Iowa Engineering Experiment Station has developed 13 "mortality type curves," which cover the entire, or nearly the entire, field of mortality characteristics of physical property Four of these are in the "left mode," five in the "symmetrical mode" and four in the "right mode" groups, in which the grestest annual retirement rate occurs respectively *before, at* and *after* ages equal to "average" life In making depreciation determinations, the proper type curve fitting the class of property in question is selected and used

Figure 2 shows the 5 "symmetrical mode" mortality type curves

Figure 3 shows mortality type curve "S_3," the middle curve of the symmetrical mode group

For each mortality type curve, expectancies have been computed by

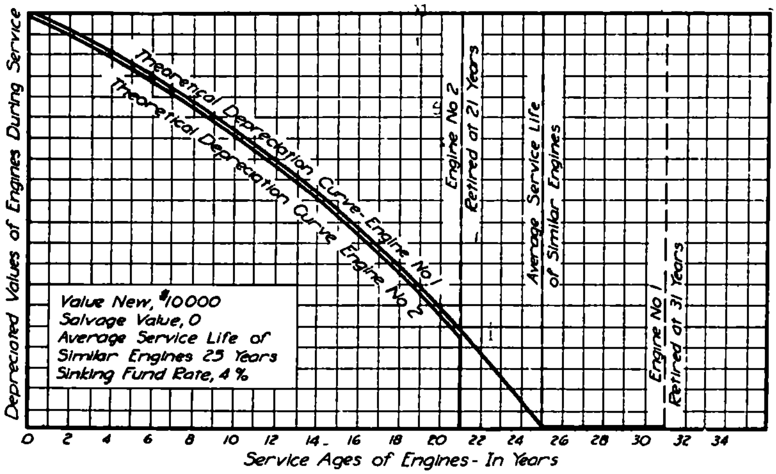


Figure 5. Depreciated Values of Two Duplicate Pumping Engines During Service Showing Theoretical Depreciation By Sinking Fund Theoretical Depreciation Method.

Form I — Physical Property Record

SAC COUNTY ELECTRIC COMPANY, SAC CITY, IOWA

Unit: Permutit Water Softener — Location: Power Plant Building — DIVISION: Sac City Power Plant
Manufacturer: Crane Mfg. Co. Chicago, Ill — Date Installed: Oct. 19, 1929 — ACCOUNT: 315 Miscellaneous Power Plant Equipment
Purchased From: Mid-West Supply Co. Des M. — Rating or Size: 12,000 gallon per day
Mfg's Serial Number: 341289-12 — Type Kind, etc: R-4 — ASSIGNED NUMBER: P 321

DATE (1)	NUMBER IN SERVICE (2)	ITEM (3)	COST STORES (4)	INSTALLATION COSTS (5)	TOTAL COST INSTALLED (6)	ESTIMATED NET SALVAGE VALUE (7)	DEPRECIATION: AGE IN YEARS (8)	P S S IN YEARS (9)	COND PER-CENT T % (10)	P F O B R (11)	DEPREC TIVE VALUE NEW OF SURVIVORS (12)	ANNUAL DEPR ALLOWANCE (13)	DEPRECIATION RESERVE (14)	PRESENT OR RESIDUAL VALUE (15)
10/19/29		Work Order # 306												
	1	12,000 Gallon Permutit Softener	4290 00	310 00	4600 00									
		Piping:												
	10'	6" C I B & S	9 65											
	1	6" T Bell	11 00											
	1	6" Gate Valve Bell	24 00											
	178'	4" C I B & S	91 60											
	4	4" 90 Degree Bell	19 70											
	1	4" 45 Degree Bell	4 60											
	1	4" T Bell	7 70											
	2	4" Gate Valve Bell	24 00											
	1	4" Float Valve Bell	60 00											
	1	4" x 4" x 2" Y Bell	6.50											
	1	4" x 4" x 2" T Bell	7 15											
	20'	2" C I B & S	3 60											
	1	2" 90 Degree Bell	4 60											
	1	2" Gate Valve Bell	7 50											
	22'	2½" Steel Pipe Thru Reservoir	8 15											
	2	2½" 45 Degree Elbows	1 70											
		Total Piping	291 40	104 00	395 40									
10/19/29	1	TOTAL WATER SOFTENER	4581 40	414 00	4995 40	0					$ 4 995 40		$	$4 995 40
12/31/29	1	Allocated General Overhead (6 1 %)			304 72						5 300 12			5,300 12
12/31/29						0	0	15	100 0	100	5 300 12	—	—	5 300.12
12/31/30						0	1	15	96 0	100	5,300 12	212 00	212 00	5 088 12
12/31/31						0	2	15	91 6	100	5,300 12	222 61	434 61	4,865 51
12/31/32						0	3	12	82 0	100	5 300 12	519 41	954 02	4 346 10
12/31/33		(City Water Softened)				300 00	4	12	75 2	80	5 000 12	1038 03	1 992 05	3 308 07
12/31/34						300 00	5	12	67 9	80	5,000 12	1310 53	3,302 58	1,997 54
12/31/35						300 00	6	12	60.0	50	5,000 12	197 50	3 500 08	1 800 04
12/31/36						300 00	7	13	57 0	60	5,000 12	~~810.00~~	3 [illegible] 06	[illegible]
12/ 1/37		Work Order # 2176												[illegible]
	1	6" Gate Valve Bell (Retired-Broken)	24 00	10 56	34.56						4,965 56		3,[illegible]65 62	[illegible]
	1	6" Gate Valve Bell (Replacement)	20 00	10 00	30 00						4 995 56		3 [illegible]65 62	[illegible]
12/31/37	1	Allocated General Overhead (8 0%)			2 40						4,997 96	66.07	3 [illegible]65 62	[illegible]
12/31/37						300 00	8	13	49 1	60	4,997 96	[illegible] 04	3,525 56	1 772 40
12/31/38						200 00	9	12	33 0	60	5,097.96	563 00	4 088 56	1,209 40
12/31/39						200 00	10	12	22 8	60	5 097 96	312.00	4 400 56	897 40
12/31/40						200 00	11	12	11 8	60	5,097 96	336 46	4 737 02	560 94
10/ 8/41		Work Order # 3063												
		Entire Unit Retired and Salvaged									5,197 96	460 94	5,197 96	100.00
		Sold to John Blank and Co. Chicago, Ill				100 00								
						$100 00					$ 5 197 96		$ 5 197 96	0 00

SAMPLE FORM I

This form sheet for Physical Property Record is suggested for use in the accounting for simple units by the "actual present worth" depreciation method. A simple unit is defined as one not widely divisible for accountancy purposes into sub-unit items. Provision is made to accurately record the original costs, depreciation, depreciation reserve and present value of the unit over its entire service life. Large maintenance and repair items or costs can be entered in column (5) so that they are available in applying the "wise retirement formula"

Figure 6

methods like those used by life insurance actuaries, and "probable service life" tables and curves prepared The probable life curve in Figure 3 is an example Also, "frequency" curves, "probable annual renewals" curves, and "average condition per cent" curves as shown in Figure 3 (and corresponding tables) are completed or are in process of completion

A technique has been developed for making actual examinations of property units (without which "actual" depreciation cannot be determined) and of applying the observed data in determining their respective depreciations

Figure 4 shows the results of the application of the modern methods in determining the actual depreciations at different service ages of two entirely similar pumping engines, one of which proved to last 4 years less and the other 6 years longer than the *average* life of similar engines in service

Similarly, Figure 5, shows how the application of "theoretical" depreciation assumptions to the same two engines would give incorrect depreciation determinations, and fail to check out correct at the actual dates of retirement

Finally, Figure 6, shows the latest standard form developed at the Iowa Engineering Experiment Station for the complete property ledger account of a property unit throughout its entire life Similar forms are being developed for the "group" depreciation accountancy of age groups of like units

DISCUSSION

ON

VALUATION OF HIGHWAY SYSTEMS

PROF B D GREENSHIELDS, *Denison University* It seems to me that the rule that all factors affecting must be given due consideration has led to much uncertainty and litigation For this reason would it not be better to use fewer factors in attempting to evaluate a highway system than are used in evaluating other public utilities

The value of a utility based upon the investment cost may be quite different from the value shown by the annual return, the original cost less depreciation may be at variance with the reproduction cost less depreciation Would it not be best to use, say, only the original cost less depreciation than to attempt to bring in other elements which may be conflicting?

DEAN MARSTON By the ruling of the Supreme Court all factors affecting value must be taken into account and each of them must be given such weight as is just and right Original costs are not only to be considered—the reproduction cost is not the only thing In the case of public utilities, earning value can be given little or no weight since earnings are what are to be regulated, but service-worth value is the value which would be based upon the reasonable worth of the services that must never be exceeded I think the same Smyth v Ames rule applies and should apply to highways There are lots of chances to use your own judgment There are some roads in Iowa, also some road structures, that are not worth what they would cost to reproduce The culverts and grades about to be deserted in making alignments are not worth anything

MAKING AND USING THE TRAFFIC CENSUS

By E W James

Chief, Division of Highway Transport, U S Bureau of Public Roads

SYNOPSIS

Owing to the present day variety of uses for traffic data the making of a traffic census is a much more complicated operation than the simple count every thirteen days that was started by France in 1844 They are used as bases for construction and maintenance programs, for selection of type and design of pavements, for segregation of routes, for apportionment of revenues, for common carrier rate making, for determining relations between motor vehicle taxes and general property taxes and for many other purposes The cost of a survey planned to yield information on all of the problems is ordinarily prohibitive, so careful advance study is necessary in order that a survey will be certain to yield the particular data wanted Suitable comprehensive forms must be arranged and "details of the master schedule and field organization must be devised with the rigidity necessary to produce regularity of observation, adequacy of supervision and speed in filing field reports, checking for possible errors, and general supervision " However, it must be possible to make quick changes to care for unforeseen contingencies

Since the daily traffic at a station is computed from observations made upon comparatively few actual days during a year and for only a part of those days, it is important that these samples be sufficiently representative to make possible an adequate estimate of the mean daily traffic volumes If the annual traffic is homogeneous and distributed according to the normal probability curve, the standard error in the mean daily traffic can be determined by statistical methods Whether or not traffic is so stable and homogeneous as to afford correct results by statistical analysis has not been fully demonstrated, and therefore studies of such complete statistical universes as those provided by the yearly records of toll bridges or tunnels are advocated

As the French were the leaders both in the art of modern road building and in the science of highway engineering, it is not strange that they should have been the first to develop a traffic census Sometime prior to 1844 the first census was taken, for by that year the French had devised the simple and effective schedule that has characterized practically every census since made, and from that time at least until the World War of 1914 there has been a regular periodical census on the national routes of France

The French schedule required an observation at intervals of thirteen days, which advanced the record one week day in each fortnight, gave a four weeks' record each year, and distributed this record uniformly over each of the four seasons The natural ratio of observations was retained for each day of the week, although it is probable that week-end traffic was not so different from the rest of the week day traffic as we find it to be Probably market days rather than Saturdays and Sundays were, under conditions existing in France in the middle and late decades of the Nineteenth Century, the days showing the widest variations from the daily average

The three elements in the French system that are noteworthy are the regularity of the observations, the distribution to include all days, and the care taken to maintain the natural ratio among the several observations

The French appear to have used their data for a single purpose—to determine the probable wear and consequently the amount of stone required for maintenance purposes On this determination depended the estimates for annual maintenance costs

The first use of the traffic census in the United States appears to have been made about 1885 The first systematic census was probably not taken until 1906 when Dean A N Johnson made a study of horse-drawn vehicles in the State of Illinois Since that time the traffic census has had more frequent use

The first systematic study in the United States made according to the methods developed by the French engineers was undoubedly that undertaken by the Bureau of Public Roads in 1911 in connection with its studies of the cost of maintaining different types of surfaces used in the Chevy Chase experimental roads and other experiments in the vicinity of Washington, D C In these observations the French system was used practically without change and some attempts were made to evaluate the data in terms of weights The presence in the traffic of a large fraction of motor vehicles made such attempt more or less futile

Since that time the traffic census has come into more or less general use in the United States, and it is interesting to note the variety of purposes to which it has been put

Studies of the density of traffic serve as a basis for programs of construction and maintenance, the classification of roads or routes, the determination of economy of types, for forecasts of future traffic, and for establishing necessary widths

Classification of traffic by types of vehicles furnishes data for the selection of types of pavements, the determination of widths, the segregation of routes for commercial or pleasure purposes, general cross section design, apportioning motor vehicle revenues, and adjusting reciprocity conditions among the several states

The classification of traffic with respect to the kind of operation will serve as a basis for fixing common carrier rates, for determining tourist requirements, for adjusting connecting routes, for establishing conditions of reciprocity, and for determining the relations of gas and motor vehicle taxes

Further studies classifying traffic as to the situs of its origin may be used in the determination of the relations between motor and general property taxes for roads, for classifying roads as of general and of local use, for allotting funds for road construction, and for the general determination of highway financial policies

Obviously, no one survey of traffic has ever been applied to all these purposes, and it is equally obvious that the simple French enumeration and classification by collars will not serve for so varied an analysis as this list of uses indicates The variety of uses listed at once discloses that in any census the details of the record taken must include such data as may be necessary and sufficient to answer the queries raised Expansion of the field data to this necessary and sufficient scope may increase the cost of the traffic census to unwarrantably large figures For this reason, it is essential in the interest of economy to determine very definitely what is wanted from a proposed survey and to organize the field work and devise the record to produce the data as inexpensively as possible This appears at first glance as a simple thing to do But it may be found at the time analysis is undertaken that some leak or loophole in the data exists and it is generally not possible to supply the omission The only certain way to determine exactly what form the record shall take in any case is practically to go through a moot analysis in advance and cover every detail required

In a recent study made by the Bureau of Public Roads of what appeared to be a simple problem, every point raised by the preliminary study of the case was successfully and adequately covered This analysis was sufficiently long to require 56 separate steps in the form of columnar calculations, and each step had to be anticipated and provided for But, in the course of the analysis, the data revealed certain facts that led to new and different queries Attempt to answer these queries with the data assembled and entirely adequate for the original purpose as outlined developed a single omission in the material necessary to answer the question This omission cannot easily be filled, and the circumstance indicates how essential it is that the full case be stated and studied in advance of any traffic survey and the field work organized accordingly.

The case referred to above is the delay study made of the viaduct across the New Jersey meadows The data seems to point to an interesting condition with respect to heavy truck traffic which was not thought of at the time the study was planned Heavy truck traffic appears in a lower ration to all truck traffic on the viaduct than it did on the surface roads Is this or is this not an indication that the reduction of congestion and the level grades on the surface roads have operated to attract and retain heavy truck traffic, which thus avoids the five per cent ramps and the three per cent grades a thousand feet or more long that exist on the viaduct The factual material to answer this very interesting query was not at hand—in fact, only a single detail was lacking—although the original data supplied everything else for the analysis as planned The unexpected query went just beyond the scope of the data

With the required data defined it next becomes necessary to prepare suitable forms for the record These sheets, which used to be a small card or at most a letter size page, are now sometimes of cumbersome dimensions Surveys recently conducted by the Bureau have used sheets as large as fourteen by twenty-six inches

To handle such field records requires usually a lapboard to which the sheets may be attached with rubber bands or metal clips, and frequently such boards are equipped also with the necessary clickers or registering tallies to carry the heavier counts being made

To discuss the various forms of information needed to supply answers to the queries implied in the list of possible uses of the traffic census already cited would go far beyond the scope of this brief paper It will be interesting rather to note certain devices used to simplify securing a good record in a survey that is so large or so far flung in the area covered as to be difficult or so involved with respect to required data as to make it desirable to subdivide the record

It is first to be noted that regularity is one of the essential features of the record Because of this a master schedule of observations is feasible and useful This master form can then be applied to any number of stations To plan the master schedule it is necessary to know the length of the proposed cycle of observations This cycle should be one more or one less than a multiple of seven, so that each succeeding cycle will start on a different day of the week In the total cycle must be included the customary idle days of the observers These idle days can usually be made to average one day off in seven, but they cannot come at regular intervals of seven days The difference between the total length of cycle and the idle days, represents, of course, the number of stations that an observer or an observing party can cover

A simple form for this master schedule is possible and once made it can be applied to as many separate observing groups as may be necessary in the survey It further serves as a fully adequate basis for correlating any miscellaneous observations that the proposed analysis requires, and its principal advantage is that new field schedules for a group of observers however numerous can be prepared based on the master schedule in a few minutes, whereas the older method of making up a schedule for each observing party independently took considerable time, and sometimes led to confusion

The master schedule is related directly to the field organization and indicates its form A concrete example will serve to indicate what this relation is In a traffic survey in New Jersey recently completed, the state was divided into three sections In each section there were sufficient stations to require three observing parties for day time counts, one for night counts, one special truck party taking special data, and one special bridge party which covered the bridges along the Hudson and Delaware Rivers The size of these divisions was determined by the number of stations in the master schedule, and over each division was placed a supervisor who had charge of the six observing parties operating in the division In all, some 400 men were used on the New Jersey survey because of frequent changes of personnel to spread employment To maintain a carefully observed schedule under such conditions demanded close supervision

The field arrangements were further simplified by an orderly designation of stations All stations in the same cycle were numbered beginning with 101, 201, 301, et cetera, just as houses are numbered 100 to the block The truck stations were numbered from one to 79, and the bridge stations from 80 to 100 In the record the kind of station and its general location, the party operating it, and the supervisor in charge are all disclosed at a glance

These details of master schedule and field organization, while simple and plain, must be devised with the rigidity necessary to produce regularity of observation, adequacy of supervision, and speed in filing the field reports, checking for possible errors and general supervision At the same time, it must be possible to make quick changes if some eventuality indicates such to be needed, and the complete control furnished by the master schedule has appeared, since its use was begun, to offer such possibility whenever tested

Another new device which has been introduced, as traffic surveys have become more varied and detailed, is the station summary sheet. It was the old practice to file observers' reports as they came in, grouping them by stations and comparing them with each other to detect internal evidences of inaccuracy or carelessness. This was done in the head office at Washington Now there is used a summary sheet in the field office, on which the complete record for each station is entered as the report is received from the field There are rarely in any survey more than 12 to 15 parties and this means not more than that many reports to check and enter daily Usually it takes less than the time of one clerk to do this

Spreading the record on a ledger or summary sheet in this way at once enables the manager of the survey to check the proper date and hour of the observation with respect to the predetermined schedule. Setting up the successive counts for each station in parallel permits an instant comparison and the detection of any unusual condition of the record. In this way it is possible to check derelictions on the part of the observers quickly and often to secure correction of records that otherwise would be lost.

It is further possible on these summary sheets to make all daily summations, and to use the sheets at the conclusion of field work for the general summations and averages for each station. This represents a considerable initial step in the analysis.

In the use of the summary sheets it is to be noted that the work is entirely mechanical—one of accurately transcribing a record. Accuracy is absolutely essential in this process. For, if a false entry is made it cannot be detected except by a general repetition of the work. Such a check would void the whole purpose of this device.

Turning attention now to the use of the census studies, we are confronted with details of statistical analysis that are interesting. If we take fourteen 8-hour observations on a station, which is the number permitted by a 27 day cycle, the total sample for the station is approximately 1.3 per cent of the annual total. This small sample can only be justified if we are dealing with conditions that have fixed characteristics, with a subject or universe that is homogeneous. For instance, if every tenth man passing a point on Pennsylvania Avenue, Washington, D. C., in a single day were carefully measured for height, the average of the measurements would be a very close approximation of the average height of all male citizens in the United States, although the sample would not be greater than eight one hundredths of one per cent. This is so because the subject is homogeneous. In the worst cases the extremes are not remote, and large groups fall within very narrow limits. The actual sport cases are so few, they go into museums. Similarly with most biological data. It is homogeneous and we can arrive soundly at the average size of a mature oak leaf, the average number of kernels on an ear of a certain variety of corn, the protein content of wheat; et cetera, with some very substantial assurance that the averages represent something worth while—an approximate truth so close to the real truth as to be usable for solving related problems.

The percentage of the statistical universe that is included in the sample is not important as a factor in the accuracy of the estimate. It is important, however, that the sample be sufficiently representative to make possible an adequate estimate of the mean daily traffic volume and samples can be representative only of homogeneous totals. If the samples be taken at random, or better than at random, throughout the universe, which is usually taken as a year, and if this universe is distributed in the form of a normal curve or a distribution more concentrated than the normal curve, then the standard error of the means is equal to or less than the standard deviation of the items in the sample, divided by the square root of the number of the items less one

This law indicates that the larger the number of items in a sample, the smaller will be the standard error If the number of items in a sample be as few as 14 (as is true of present practice) the standard error of the mean would be equal to the standard deviation divided by 3 7; but, if the number of samples be as many as 70, the standard error would be equal to the standard deviation divided by 8 3. Such a larger number of long counts would make the expense prohibitive, and it is, therefore, necessary to reduce the length of the watch The possibility is suggested that the watch might be reduced from 8 hours to 30 minutes, or less, in order that the number of watches may be increased while the total time required for the survey may be reduced. The Bureau is now investigating the effect of this and its practicability in field operations

Now, the very interesting question confronts us in all traffic analysis as to whether traffic is certainly so stable and so homogeneous in its character as to permit of statistical analysis with any assurance that our figures are valid Few of us have ever stopped to consider this in the past We have assumed that our traffic census results are probably within some 10 per cent correct, which is a satisfactory working approximation of the truth Actually, nothing specific has occurred to indicate definite error in past results, and traffic census studies have been so scattered and simple in most cases that no indication has been given to make us doubt the soundness of traffic statistics honestly taken and analyzed according to current methods

In fact, such tests as we have been able to make have supported our position in the rough The check survey conducted by the Bureau and the State Highway Department in New Hampshire, after an interval of exactly five years, showed results close enough to satisfy practical requirements Some studies made by the Port of New York Authority satisfactorily stood the test of extrapolation backward to apply to old records Studies of monthly daily average traffic on many surveys indicate a decidedly characteristic annual curve—a flattened sine curve with the line of the mean producing intersections almost always within a spread of two weeks in April–May and October–November

In spite of these apparently assuring circumstances we are faced with the fact that traffic surveys are becoming more and more detailed, the national economy as affected by highway construction and highway transport is demanding more and more data, and more exact data, analysis is becoming complicated and lengthy in keeping with the increased and more elaborate field data, and, of course, with it all goes a serious increase of cost The net result is a growing advocacy of abbreviated surveys, which will first of all reduce costs and error, and as a supplementary advantage reduce the time required to make the survey and the analysis of the data

We cannot refuse to accept the soundness of the advocates of skeleton surveys from the point of view they have taken The natural question is, however, as to how such abbreviation will affect the accuracy of our studies All we have now with which to compare the analysis of a skeletonized survey is the analysis of another more elaborate survey in the same area One result will perhaps differ from the other But we cannot tell whether that difference represents an increase or decrease in accuracy, nor do we know what the error is in our present method, because we are making the fundamental assumption of a homogeneous subject of investigation, where in fact we may not have that condition in large degree

If, therefore, we are to continue using elaborate and more elaborate traffic data, as is certainly indicated by existing requirements, and if we are to reduce the time spent on the surveys below present practice, we should institute some investigations that will be helpful in determining the general approximation under which we are now working and the effects of skeletonizing our present methods

To do this we must have a complete statistical universe to work with We could get this by taking a record for 24 hours a day throughout a year Such an experiment is out of the question, besides, we wish to have an answer to this question sooner than such a method will permit

The necessary steps toward a study of the soundness of our present methods and an actual determination of their accuracy are clear The determination of the relative accuracy of derived or abbreviated methods must be the analysis of full records that are now in existence, such as those for toll bridges or tunnels, which constitute the necessary complete universe in a statistical sense Against these observed totals others computed from any variety of sampling may be set in comparison, and by these means we will be able to determine just how far we may advisedly go in changing our present methods The mathematical theory of sampling points toward more but shorter watches with a gain in precision and a saving in expense, together with more prompt results Will complete traffic records lend themselves to the analysis of mathematical method? Will the means estimated from a considerable sample of short counts prove to be nearer the true mean than those estimated from the smaller samples of longer counts now in use? Theory answers "Yes" But, we must test our theory by its practical application

DISCUSSION

ON

TRAFFIC CENSUS

MR BURTON MARSH, *American Automobile Association* It might be interesting to point out results of a limited analysis of this same matter made in Philadelphia in 1932 Eight hour traffic counts (7–11 A M and 2 30–6 30 P M) had been made for some 2,000 intersections Also at several key stations, some 16 hour and some 24 hour counts had been made It was realized that the man power required for keeping such a record up to date, year after year, was quite out of the question Therefore, our Traffic Statistician analyzed the records for different one and two hour periods and for different types of districts He found that there were differences throughout the hours of the day dependent upon the type of district After many attempts to work out satisfactory factors, the study was stopped without a successful outcome However, this analysis could *not* be considered as *definite* proof that satisfactory short-time count factors cannot be developed for a city like Philadelphia, for some of the counts were perhaps not accurately made Many of the counts were made by unemployment relief workers, and while there was quite close supervision, it was probably not adequate at some intersections I am confident that the analyses were sound, as they were made by a very competent statistician

This illustration *does* indicate one of two conclusions (a) There are too many variables in a city like Philadelphia to develop satisfactory short-time count factors, or (b) The original field counts were not accurately made (For example, the men may not have shifted their tallying at exactly the end of the specified half-hour periods, there may have been dishonest tallying, or certain observers may not have been able to "keep up" with the vehicular flow, and may have "estimated" certain heavy flows) If this is true, it emphasizes the necessity of strictly accurate tallying as the basis for determining short-time count factors

DETERMINATION OF TIME SAVED BY VEHICLES USING THE NEW JERSEY HIGH LEVEL VIADUCT

BY LAWRENCE S TUTTLE

Assistant Highway Economist, U S Bureau of Public Roads

[In Abstract*]

This paper presents an estimate of the vehicle time saved annually by the recently completed viaduct between Newark and Jersey City, under present traffic conditions Field studies were made before and

* This paper was published in full in Public Roads Vol 14, No 12, February, 1934

after the opening of the viaduct by the Division of Highway Transport, Bureau of Public Roads, in cooperation with the New Jersey State Highway Commission

The viaduct completes the express highway from Elizabeth to the Holland Tunnel, which is one of the most heavily traveled routes in the United States Traffic congestion on the old ground level route often reached serious proportions due to frequent opening of draw bridges and traffic at numerous intersections Traffic counts indicated

DAILY TRAFFIC BEFORE OPENING OF VIADUCT (OLD ROUTE)—AT HACKENSACK RIVER BRIDGE

	Passenger cars	Light trucks	Heavy trucks	Total
Week days	28,970	3,780	4,650	37,400
Saturdays	35,710	2,385	2,705	40,800
Sundays and holidays	47,070	1,377	853	49,300

DAILY TRAFFIC AFTER OPENING OF VIADUCT (OLD ROUTE)

	Passenger cars	Light trucks	Heavy trucks	Total
At Hackensack River Bridge				
Week days	11,290	1,976	3,334	16,660
Saturdays	13,830	1,176	1,994	17,000
Sundays and holidays	9,940	352	308	10,600
Traffic diverted to the viaduct				
Week days	17,680	1,804	1,316	20,800
Saturdays	21,880	1,209	711	23,800
Sundays and holidays	37,130	1,025	545	38,700

SUMMARY OF VEHICLE-MINUTES SAVED PER YEAR

	Minimum estimate	Maximum estimate
Passenger cars	47,407,000	57,445,000
Light trucks	3,521,000	4,833,000
Heavy trucks	3,744,000	3,827,000
Total	54,672,000	66,105,000

a total yearly traffic during 1932 at the Hackensack River bridge of 14,600,000 vehicles, with an average week-day traffic of 37,000 vehicles and average Sunday traffic of 50,000 vehicles

The trip time for vehicles on both the old and new routes was obtained by listing the vehicle license numbers at both ends of the route together with the time of passage to the nearest minute Vehicles were classified as passenger cars, light and heavy trucks, light trucks being 2½ tons or less capacity

The viaduct cost $19,300,000 Capitalizing this amount at six per cent would require the annual savings to be $1,158,000 This would place a value of 1 75 to 2 12 cents on each vehicle-minute saved It is emphasized, however, that these estimates do not include all savings which may be credited to the viaduct There is a saving in travel distance and also an additional volume of traffic which previously avoided the old route on account of the congestion Traffic is expected to increase to a greater amount than the old route could have served

In discussion, Mr F LAVIS, *Consulting Engineer, New York* said

I think the value of these economic studies and their effect on the design cannot be too strongly emphasized and it seems advisable to further stress the fact that such studies have two distinct phases

First, the phase which governs the design of the *route*, that is, studies of the effect of the route, its length, freedom from delays, rise and fall, curvature, etc , on the operation of the vehicles which use it

Second, the phase which governs the design of the *structure* which carries the route

These two studies, while in a certain sense entirely separate and distinct, are in many other ways interdependent and must necessarily be considered together

The second of these two phases has been ably treated in a paper[1] by Mr S. S Johannesson recently presented to the American Society of Civil Engineers The first was set forth in my own paper[2] presented to the same Society some three years ago

It is the first of these two phases to which Mr Tuttle calls attention in his reference to the economic problems which controlled the design of the route He has, however, confined his present studies to only one section of the project, that from mile 1 4 to mile 4 7, out of a total route length of about 13 miles and, further, these studies have been confined only to the economic value of time savings and do not take into account, except by passing reference, the not inconsiderable economic gain due to the savings in distance and other factors

Even on this particular section of 3 3 miles the saving in distance is some 4,000 ft which, from the character of traffic using this highway, indicates (see my paper above referred to) a justifiable expenditure of some $25,000,000 to $30,000,000 or considerably more than its actual total cost

Mr Tuttle is careful to point out that there are these other factors but I think there is some danger that his conclusion, "that in order to justify the construction of the Viaduct a vehicle minute must be

[1] Lincoln Highway, Jersey City to Elizabeth, N J Proc ASCE Nov 1933, p 1389

[2] Highways as Elements in Transportation F Lavis, Transactions ASCE Vol 95 (1931), p 1020

valued at 2.12 cents or 1.75 cents" may be lifted from its context, quoted elsewhere, and be thus misunderstood. It may, I fear, convey the inference that this structure or this project had not sufficient economic justification in as much as these values are too high.

I think that further attention might well be called to the fact, which is also parenthetically mentioned by Mr. Tuttle, that economic justification of any route of this kind should be based on reasonable anticipations of traffic, or traffic capacity, and not necessarily on the traffic which may actually use it during the first few years after it is placed in service.

It would seem also that certain highway engineers have not yet entirely realized the fact that it is the capitalized value of the annual savings which determines the justifiable expenditure.

It is important also to bear in mind other characteristics of this whole 13 mile route which are not apparent on the Viaduct section. I refer particularly to the relief of congestion in the streets of the business, manufacturing and residential sections of Jersey City, Newark and Elizabeth.

In addition to the savings in costs of operation of vehicles actually using this route because of savings in time, distance, etc., which alone would have justified its cost, there is the vast relief and savings to the vehicles, merchants, manufacturers, industrialists and residents of these cities by taking this large volume of traffic off the streets which they require for use in their businesses and daily avocations.

It is almost impossible to estimate the money value of savings of this nature but my own opinion is that this also, by itself, would have justified the construction of this $40,000,000 project.

Unthinking people have protested against the expenditure of this large sum of money by the State of New Jersey to help, as it has been stated, a lot of cars not registered in that State. The cities of Jersey City, Newark and Elizabeth are, however, large taxpayers and furnish a not inconsiderable part of the State revenue both on the basis of assessed values of real property and in contributions of motor vehicle and gasoline taxes, and they were entitled to this relief from the almost intolerable burden imposed on their streets by this through traffic. The construction of this route is a lasting and very substantial benefit to them as well as to the actual users of the new route.

MR. S. JOHANNESSON, *New Jersey Highway Commission*, said:

There is no doubt that the results obtained give a true picture of the traffic conditions at the time the study was made, but owing to the effects of the present business depression, it is not believed that the results will represent the condition that will exist when we return to normal times.

The records of the bridge openings made during the period that the traffic studies recorded in the paper were made, indicate that their number is only about 50 per cent of that which might have been expected in accordance with the trend shown for several previous years, and the records of the traffic volume on the highway indicate that this also was materially less than that which might have been expected in normal times

In order to get a proper picture of the economics connected with vehicle operation over the new high level viaduct as compared with travel over the old highway, it is necessary to make use of figures which will apply in normal times. When business conditions do improve, it is probable that the river traffic will be materially increased, resulting in an increase in the number of times that the highway traffic has to be stopped This has an important bearing, because when the highway traffic is heavy, the total time lost is not proportional to the time the bridges are closed to vehicle traffic, but more nearly to the square thereof

It is possible to show that even at the present times, the amount saved in cost of vehicle operation by the vehicles passing over the new viaduct represents an interest on the capital invested of more than six per cent, but the actual savings will far exceed this amount when normal conditions return

The fact is that the new viaduct was built to relieve the traffic congestion that had developed on the old Lincoln Highway, a congestion that was becoming so serious that much transportation was diverted from this highway on account of the delays suffered The construction of the new viaduct is a means of saving the increase in transportation cost on this account and this saving should properly be credited to its account

MR BURTON W MARSH, *Traffic Engineer, American Automobile Association, Washington, D C*, said

It seems to me that it would be of great interest to know three things about the 50 foot roadway on the viaduct 1 How much use is being made of the middle lane? As I understand it, there are five 10 foot lanes 2 How many accidents have resulted thereby? As I understand it, this is a high speed viaduct 3 Is it considered by those present and other highway officials that a five lane width, meaning necessarily a third lane to be used only for passing or in one direction only, is a desirable arrangement for high speed arteries of that type

APPLICATION OF EQUATION FOR ANNUAL ROAD COSTS

BY SIGVALD JOHANNESSON

Designing Engineer, New Jersey Highway Commission

SYNOPSIS

The equation for annual road cost proposed in the Ninth Proceedings of the Highway Research Board is simplified by replacing the factors

$$\frac{r}{(1+r)^n - 1}, \frac{r}{(1+r)^{n'} - 1} \text{ etc}$$

by k, k' etc , which are constants depending on the length of life (n) of the item in question and on the rate of interest (r) Considering that within the probable limits of the value of r there are no serious differences in the value of k for a given value of n, and eliminating values of n less than 10 and greater than 20 he reduces the general equation to

$$C = Ar + B + 0\ 085E_{10} + 0\ 052E_{15} + 0\ 035E_{20}$$

in which E_{10} represents the estimated replacement cost of such items as have an expected life of from 10 to 14 years, E_{15} from 15 to 19 years and E_{20} from 20 to 30 years

In considering construction costs the interesting conception is offered that since highways were originally built as means of communication and in later years have also become means of transportation in some cases the costs incident to the two purposes should be separately considered

EQUATION FOR ANNUAL ROAD COST

In the report of the Committee on Highway Transportation Costs of the Highway Research Board for 1929,[1] the average annual road cost is expressed by the equation

$$C = r\left(A + \frac{B}{r} + \frac{E}{(1+r)^n - 1} + \frac{E'}{(1+r)^{n'} - 1} + \quad\right) \tag{1}$$

where

C = average annual road cost

A = cost to construct

B = yearly maintenance cost (every year)

E = (or E') = expenditure for periodic maintenance every n (or n') years (Replacement is an "E" value)

r = the rate of interest prevailing in current state financing

This equation may be simplified in the following manner

Let

$$k = \frac{r}{(1+r)^n - 1}, \text{ and } k' = \frac{r}{(1+r)^{n'} - 1}, \text{ etc ,}$$

then equation (1) may be written

$$C = Ar + B + kE + k'E' + \tag{2}$$

[1] Ninth Proceedings Highway Research Board, page 360

in which k, k' etc. are constants depending on the length of life, n, of the item of highway construction in question and on the rate of interest, r. When these two values are estimated or known, the corresponding value of k may be found in Table I.

For general purposes equation (2) may be simplified as follows.

Considering the figures in Table I it is apparent that within the probable limits of the values of r there are no serious differences in the values of k for a given value of n. It is possible, therefore, without material error to substitute Table II for Table I.

TABLE I

VALUES OF k FOR VARIOUS VALUES OF n AND r

n in years	r in per cent							
	2.5	3	3.5	4	4.5	5	5.5	6
5	0.1903	0.1883	0.1865	0.1846	0.1828	0.1810	0.1792	0.1774
10	0.0893	0.0872	0.0852	0.0833	0.0814	0.0795	0.0777	0.0759
15	0.0558	0.0538	0.0518	0.0499	0.0481	0.0463	0.0446	0.0430
20	0.0391	0.0372	0.0354	0.0336	0.0319	0.0302	0.0287	0.0273
25	0.0293	0.0274	0.0257	0.0240	0.0224	0.0210	0.0195	0.0182
30	0.0228	0.0210	0.0194	0.0178	0.0164	0.0151	0.0138	0.0126
40	0.0148	0.0133	0.0118	0.0105	0.0093	0.0083	0.0073	0.0065
50	0.0103	0.0089	0.0076	0.0066	0.0056	0.0048	0.0041	0.0034
60	0.0074	0.0059	0.0051	0.0042	0.0035	0.0028	0.0023	0.0019
80	0.0040	0.0031	0.0024	0.0018	0.0014	0.0010	0.0008	0.0006
100	0.0023	0.0016	0.0012	0.0008	0.0006	0.0004	0.0003	0.0002

TABLE II

APPROXIMATE VALUES OF k FOR VARIOUS VALUES OF n

n	k	n	k
5	0.185	40	0.012
10	0.085	50	0.007
15	0.052	60	0.005
20	0.035	80	0.002
25	0.025	100	0.001
30	0.019		

It is not likely, however, that any item of work with an expected life of less than 10 years will be placed in the permanent highway structure, and consequently the value of k for $n = 5$ may be omitted. Further, the values of k for n greater than 20 may be omitted, because first, it is scarcely possible to predict that the life of a certain item will be 20, 25 or 30 years, and it is better, keeping in mind the purpose for which the equation is to be used, to select the lowest and most conservative figure, and second, if the expected life is more than, say, 30 years, the value of k is so small that no material error is committed in omitting all items having an expected life of more than 30 years.

For general purposes, therefore, equation (2) may be written

$$C = Ar + B + 0.085E_{10} + 0.052E_{15} + 0.035E_{20} \quad (3)$$

in which E_{10} represents the estimated replacement cost of such items as have an expected life of from 10 to 14 years, E_{15} the replacement cost of items having an expected life of from 15 to 19 years, and E_{20} the replacement cost of items having an expected life of from 20 to 30 years

Equation (3) may be used under all ordinary circumstances If, however, in specific cases the interest rate may have an exceptional value, or if construction items of relatively high cost may be estimated to have an unusually long life, it is proper to refer to Equation (2) in conjunction with Table I

COST OF CONSTRUCTION

(a) If the equation for annual road cost is used in connection with the study of the economics of a projected highway improvement, the symbol A in the equation represents the estimated cost of construction of the improvement It does not include any previous expenditures on the highway

The estimated cost of construction of the improvement will include the cost of additional right-of-way, if such is required to carry out the improvement It will include also cost of damages on account of the improvement, for which payments may have to be made It will further include the estimated cost of actual construction work contemplated together with engineering and legal expenses

For a complete economic study there are other items, which should be included under the heading "Cost of Construction," such as General Office Expenses, Interest during Construction etc , but in many cases the value of these items may be quite uncertain or difficult to obtain The simplest way of including these items, therefore, may be to add to the total estimated cost of construction an amount of from 5 to 10 per cent of this total The possible error incurred will be immaterial

(b) If the equation for annual road costs is used for evaluating a highway in a study of proper taxation of motor vehicles, cost of previous construction work should be taken into account It is the opinion of the writer, however, that only cost of improvements, which were made at or subsequent to the time, when improvements were made for transportation purposes, should be considered This opinion is based on the following reasoning

There are two reasons for constructing highways One is to provide a means of communication and the other to provide a means of transportation Until comparatively recently the communication purpose was overwhelmingly predominant on all highways It is still predominant on some secondary roads, but on primary roads the transportation purpose is now of the greater importance

At the time a highway was constructed for communication purposes, there was undoubtedly little or no thought of it becoming eventually of material importance as a means of transportation Its social and economic value as a means of communication, therefore, must have been of sufficient importance to warrant the cost of its construction and maintenance from the time it was originally built to the time when it became important as a means of transportation The Appian Way was constructed for military and administrative reasons and not for transportation as we now understand this term The Boston Post Road was constructed originally as a line of communication between New York and New England, as implied by its name Other roads were built originally to serve as a means of communication and access to land and settlements adjacent to them In fact, it is almost certain that the value of all such roads as a means of communication remains, even though they may now also be used as means of transportation

It appears, therefore, that it is unnecessary, and perhaps even improper, when determining the cost of a highway for the purpose of vehicle taxation, to include in the cost determination the cost of any right-of-way, grading, structures and other items, which were provided prior to the time that the highway changed in character from a means of communication to a means of transportation These items were socially and economically justified to provide a means of communication and should not be charged to transportation

(c) If, on the other hand, the equation for annual road costs is used for valuation of the highway as a whole, it is necessary to include all the items of right-of-way and permanent construction in the manner indicated in the report of 1930.

ANNUAL COST OF MAINTENANCE

In equation (2) or (3), the term B represents the annual cost of maintenance This item includes all repairs made on the various parts of the highway structure in order to retain it as nearly as practicable in the condition it was in at the time the construction was just completed It includes also the annual expenditures for such items as lighting, attendants at tunnels or movable bridges, operating cost of machinery and equipment of tunnels or movable bridges It may include further the cost of street cleaning and snow removal The item, however, does not include additional surfacing for the purpose of widening the paved roadway, additional drainage facilities or other similar work, because such work is a new and additional improvement

Presuming that the maintenance is such as to keep the highway structure in good condition at all times, the maintenance cost will depend on the length, width and type of pavement, the volume of traffic, the climate, the geographical location and many other conditions The cost of maintenance probably will vary with the age of the surface, but for convenience of computation, the average annual cost may be used

The maintenance cost is an important item in the annual cost of the highway and deserves careful study A highway department making a study of the economic justification of a highway improvement usually will have records of previous work on which an estimate of the cost of maintenance may be based, unless the improvement contemplates types of construction items not previously used by that department To take care of this condition, which may frequently occur, it would be of material value to collect, classify and publish records of the cost of maintenance of all highway departments and other similar bodies, if such records are available for this purpose

PERIODIC MAINTENANCE (REPLACEMENTS)

The replacement items E, which may enter the equation for annual road cost may be determined by considering all the various construction items of the highway These items may be enumerated as follows·

a Right-of-Way
b Grading
c Shoulders, Curbs, Sidewalks
d Surfacing
e Drainage
f Culverts
g Guard Rails and Signs
h Lighting
i Bridges
j Tunnels
k Landscaping

Each of these items will be considered separately

(a) *Right-of-Way* The right-of-way is permanent Additional right-of-way may be acquired in the future to take care of traffic conditions that may develop, but this will involve a new improvement, which is not within the scope of the improvement considered Right-of-Way, therefore, is not a replacement item

(b) *Grading* Grading is permanent It is probable that the elements will continuously deteriorate the grading work, but the repairs necessary on this account will come under the heading Maintenance It is possible also, that from time to time changes will be made in the grading to take care of new traffic conditions, but that again will involve a new highway improvement, outside of the scope of the improvement considered Grading, therefore, is not a replacement item

(c) *Shoulders, Curbs and Sidewalks* Unpaved shoulders may be considered permanent with proper maintenance If at a later time they are paved to take care of new traffic conditions, this will be a new improvement, not within the scope of the improvement considered Unpaved shoulders, therefore, are not a replacement item Paved shoulders will deteriorate in time and may have to be rebuilt Paved shoulders, therefore, is a replacement item for which the lifetime must be estimated The same applies to curbs and paved sidewalks

(d) *Surfacing* Almost any kind of surfacing will wear out in time this, therefore, is a replacement item for which the lifetime must be estimated To facilitate this estimate it would be of material assistance to collect, classify and publish records of the actual lifetime of various types of surfaces under various traffic, climatic and geographical conditions

(e) *Drainage* The drainage system of a highway, including ditches, pipes, inlets, manholes, etc , may usually be considered permanent Damaged sections of pipe, broken grates etc , can be taken care of under the heading Maintenance If the drainage system for any reason should be found inadequate, or should fail in part on account of heavier loads being carried on the highway than originally contemplated, the reconstruction cost of such work should be charged to a new improvement and not to periodic maintenance Drainage, therefore, is not a replacement item

(f) *Culverts* Usually culverts may be considered permanent In special cases, where either the material used or the soil conditions may limit their life, they may be considered as a replacement item, for which the lifetime must be estimated

(g) *Guard Rails and Signs* As guard rails and signs will deteriorate, they must be considered as replacement items, for which the lifetime is to be estimated New or additional guard rails or signs, or changes made on account of subsequent developments should not be considered here, as they are actually new improvements, but for convenience, it may be proper to consider them as maintenance items

(h) *Lighting* It is probable that in most cases the cost of lighting highways, if any, is an annual charge, which may be included under the heading Maintenance If otherwise, it will be necessary to classify Lighting as a replacement item and estimate the lifetime of wires, cables, poles, fixtures, etc , entering into the system

(i) *Bridges* Some bridges, such as wooden trestles, have a limited life and are, therefore, a replacement item Other bridges, such as steel and concrete structures, should last indefinitely, as far as the main structure is concerned, if properly maintained The roadway and sidewalk surfaces, of course, are replacement items, and the railings and other minor items of the bridge also may have a limited life, which has to be estimated The same applies to the machinery and equipment of movable bridges Bridges which have to be rebuilt on account of obsolescence, such as those which may have to be reinforced because of greater loads than originally contemplated, or which may have to be widened on account of increased traffic, are not to be included under the replacement items, because their reconstruction involves additional improvements

(j) *Tunnels* Where tunnels occur in highway construction, they may be considered permenent, if built as permanent structures If a wooden lining is used, this will have to be replaced in time, and the wooden lining, therefore, must be considered as a replacement item, for which the lifetime must be estimated. The same applies to the roadway surface, to pumping, lighting and ventilating machinery and equipment and possibly also to other minor items of the tunnel

(k) *Landscaping* This item includes such matters as covering slopes with grass, planting trees along the highway and other similar work done for the purpose of securing and beautifying the highway Work of this character should be considered permanent and not as a replacement item Repairs, replacements and renewals are properly to be considered under the heading of Maintenance

Summarizing, the replacement items may be listed as follows

Paved shoulders, curbs and sidewalks
Roadway surfaces
Culverts under special conditions
Guard rails and signs
Lighting system under certain circumstances
Bridges, certain items only
Tunnels, certain items only

It is apparent from this list that under ordinary circumstances paved surfaces are the most important replacement items, and that, except under special circumstances, all other replacement items may be omitted without material error It is advisable, however, in preparing an estimate of this kind, to consider all the items of the list to ascertain whether or not any of the other items may be of sufficient importance to warrant inclusion in the estimate

RATE OF INTEREST

Equation (1) contains the term r, which has been defined as the rate of interest prevailing in current state financing Theoretically, this may not be entirely correct, because it may be a question whether it is proper to use the same interest rate for the money expended on the construction work and for the money set aside for the replacement work The interest rate used for the money spent on construction work may properly be considered as the interest rate prevailing in current state financing, but the interest rate of money set aside to provide a replacement fund may be different, due to legal or other limitations

Actually, however, this condition does not exist, because it is improbable that any highway board sets aside a sum of money at the time of construction of a certain highway, the proceeds of which are to be applied to future replacement Usually the money required for such replacements is derived from the current funds of the highway board The limitations that may exist on the interest rate of money invested by the highway board for future use, therefore, are not exercised, and it appears proper under the circumstances to apply the same interest rate to the replacement work as that used for the primary construction work

DISCUSSION

ON

EQUATION FOR ANNUAL ROAD COSTS

PROF W W HITCHCOCK, *Michigan State College*· I would like to call your attention to the mention which Mr Johannesson has made of the effect of omitting the reproduction cost of items which are estimated as having life of thirty years or more

By a study of Table I, it will be observed that there is a definite relation between interest rate and the value of the Constant, "k" for any fixed time period

The annual interest charge against any portion of the highway investment is an important item in the computation of the annual cost of a highway The annual deposit to reproduce this same portion of the highway then bears the same relation to the interest charge as the value of "k" bears to the interest rate To illustrate. Assume an interest rate of four per cent and the life of the improvement as 30 years If replacement cost is omitted the difference or decrease in the computed annual cost is to the interest charge as the value of "k" is to the interest rate or equal to 44 5 per cent of the annual interest charge on the same investment

By making use of this simple relation one may determine what effect the omission of the replacement charge will have on any highway in question

MR. W R COLLINGS, *Dow Chemical Company*: In most of these discussions of the economics of highways and of annual road costs, there is no provision made for payment of the initial investment Should there not be an item included to retire the bonds in a period of say 15 years?

PROF R L MORRISON, *University of Michigan* The item to retire the bonds is included in the term, or terms, covering depreciation In all of the formulae for annual road cost the three items of interest, depreciation and maintenance occur in one form or another In order to make actual payments conform to the theoretical annual cost, the construction must be financed by means of bonds

In the case of Mr Johannesson's formula the term "Ar" would be the annual interest payment on the bonds, "B" would be the actual annual maintenance cost, and the sum of the terms "$kE + K'E' +$" etc would be the annual deposit in the sinking fund

Of course no results obtained by the use of a formula can be any more accurate than the basic data used in the computations In the case under discussion we must assume the interest rate, the annual maintenance cost, and the economic life of each part of the highway, all of which are subject to considerable variation

Considering the difficulty of an accurate determination of these factors, it has seemed to me that, for practical purposes, the simple formula $C = D + I + M$, in which "D" is annual depreciation, "I" is annual interest, and "M" is annual maintenance cost, is about as satisfactory as the more elaborate formulae Also I see little use in trying to foresee the future beyond the estimated life of the original road surface Therefore I would first estimate the residual value of all parts of the highway at the end of the period covered by the estimated life of the surface Subtracting that value from the total original cost would give the total depreciation during the period under consideration That sum divided by the estimated life of the surface would give the annual depreciation, "D." Adding the interest on the original investment to the interest on the residual value, as computed above, and dividing this sum by 2, would give (approximately) the average annual interest "I" The average annual maintenance cost, "M," would merely be estimated as accurately as possible

To make actual payments correspond with the theoretical costs, as determined by this formula, it would only be necessary to finance the improvement by means of serial bonds, having an amount equal to the depreciation during the life of the surface mature during that period The bonds retired each year, then, would be just equal to the theoretical annual depreciation, "D," and the average interest payment, over the entire life of the surface, would be approximately equal to "I" in the formula

I believe it is generally agreed that, to determine the economic soundness of a proposed improvement, interest must be charged against it even if no interest is actually paid

DEAN ANSON MARSTON, *Iowa State College* I cannot help thinking that when we get studying these things from the standpoint of maintaining a depreciation reserve, just equal to the actual depreciation of the roads, a lot of these questions will clear themselves up We do not in ordinary practice consider that the cost times the interest is a part of the annual cost of production The interest is really a part of the net return on the investment In the case of highways the net return, of which the interest is part, comes to us in the form of equitable charges for road services

In the case of an ordinary utility it is necessary for the utility to make an annual depreciation appropriation out of income to make good the depreciation each year, but they do not set it aside in a sinking fund the way this question assumes They reinvest it, and that is what we are doing with our highways out of current road funds We are making replacements and improvements on old roads and are building new roads, all of which constitute investments of the sums which we collect over and above the cost of maintaining the roads

ANALYSIS OF ROAD COST ON THE STATE HIGHWAYS OF WORCESTER COUNTY, MASSACHUSETTS

By C B Breed

Professor of Railway and Highway Transportation, Massachusetts Institute of Technology

SYNOPSIS

The report presents the methods used in analyzing road costs on the state highways of Worcester County, Massachusetts The complete analysis will include about 300 miles of state highways, which in their physical layout and traffic densities constitute a traffic pattern, such as has been suggested by the author as a unit for studying road costs

Complete data are included for one typical continuous route, comprising 27 miles of state highway in 23 sections of different surface type, width or condition, and with traffic densities ranging between 870,000 and 3,700,000 vehicles per year Three tabulations have been prepared. the first giving descriptions of surfaces and annual maintenance costs, the second giving construction history and computation of capital costs, and the third summarizing the above and showing computation of annual road costs by an approximate method, and in comparison the annual contributions paid in state taxes by vehicles using the different sections

Previous reports of the Committee on Highway Transportation Economics have presented analyses of road cost for a 6-mile section of the Boston Post Road in Connecticut and a 26-mile stretch of the Des Moines-Ames road, Iowa (1930), and for the Concord-Harvard and the Tyngsboro roads (about 7 miles and 3 miles long, respectively) in Massachusetts (1932) Those studies related to isolated portions of roads having different traffic densities They were not intended to develop road costs for general use, their purpose was to develop the application of the fundamental principles set forth in the report of the Committee in 1929

The current study comprises the State System in Worcester County in Massachusetts because it constitutes a traffic pattern centering in the City of Worcester which has a population of 195,300 (See Figure 1) A statewide study could be readily developed by investigating the remaining county highway systems and assembling them Much the same problems would be encountered in studying one county as in studying an entire state The State System of Massachusetts is especially suitable for such studies because construction and maintenance costs have been kept since 1895 and traffic counts made since 1909

Worcester County was chosen because it comprises the center portion of the state and represents a fair average of Massachusetts conditions both with respect to topography and traffic density Furthermore, most types of pavement are represented in this study, they vary from waterbound macadam 15 feet wide and 35 years old to cement concrete 60 feet wide and 2 years old Traffic densities on the state highways of this county vary from 1000 to 10,000 vehicles per average day

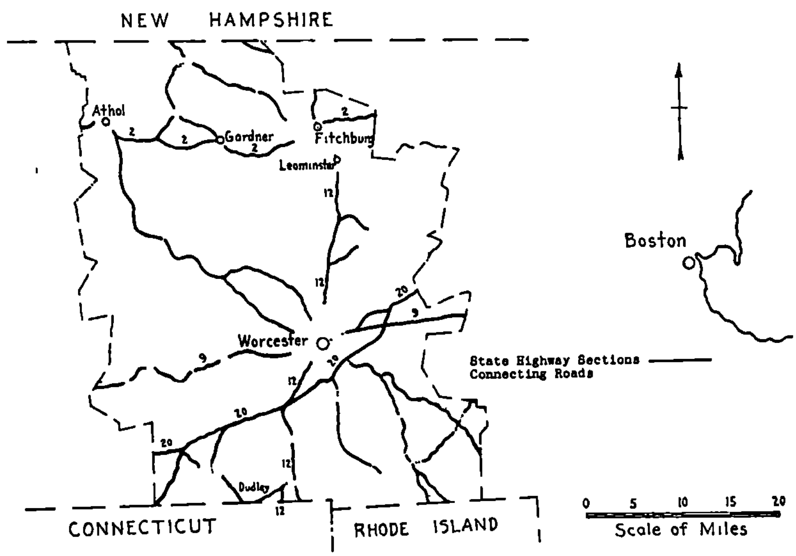


Figure 1 State Highways in Worcester County, Massachusetts

The City of Worcester is the center of a traffic pattern from which roads of heavy traffic density radiate, diminishing in density toward the county boundaries to the east, south and west where they join other patterns To the north there are minor patterns within the county, centering about Fitchburg and Gardner Worcester County has about 4300 miles of rural road of which 319 miles are state highways The portions of these routes lying within the business sections of towns and cities are usually not included in the State System and were therefore omitted from this study Route markings are of necessity continuous, including alternate sections of state and town road as indicated in road descriptions in Table I

An analysis of road cost on these town or city sections would be difficult to make, because it would be necessary to examine the records of each town or city involved In some cases reliable cost data would not be available, and in other cases the methods of accounting would vary so much between towns that costs would not be comparable There are, however, many miles of state-aid roads in Massachusetts for which cost records are available, these town roads are built by the state, but are financed partly by the state and partly by the town or county concerned In many states these state-aid roads would be a part of the state highway system, but in Massachusetts they remain town roads and are therefore not included in this study By limiting this study to state highways only, the scope of the study has been limited, with few exceptions, to heavily traveled roads, because in Massachusetts the state highway system includes only 10 per cent of the total rural mileage, upon which the heaviest rural traffic is concentrated.

DESCRIPTION OF ROADS ANALYZED

The state highway system of Massachusetts has been built up section by section over a period of 37 years and is therefore made up of great diversity of kinds and ages of pavements Within a given mile of road there are often several types of pavement While this diversity of pavements makes a study of road cost more lengthy than would be the case if, as in many states, long sections had been built at one time, yet it also offers an opportunity to compare several types of pavement under nearly identical traffic conditions

In this report a complete analysis is presented for only one typical route within the Worcester pattern Other routes may show different costs, but the method of analysis will be the same The typical route which has been chosen is State Highway Route No 12, which is described in Table I, columns (1) to (18) The surfaces are arranged in consecutive order from Leominster at the north to the Connecticut Line at Dudley to the south Nearly every kind of pavement used on Massachusetts state highways exists on this route. The portions of town or city roads are indicated in Table I where they connect sections of state highway The descriptions of pavement details were obtained from the state maintenance files and are complete only for the more recent pavements The descriptions of old pavements which are missing from Table I could have been filled in from project plans or construction records As these descriptions were considered relatively unimportant, the necessary time was not taken for this work

The data are arranged according to two route numbering systems as indicated in columns (1) and (2) The "Auto Route No ," column (1), refers to the number with which route is marked out on the road, the "Maintenance Route No ," column (2), gives the number used by the Maintenance Engineer in keeping cost and descriptive data The auto route may change from year to year, and in many places the same road

TABLE I

DESCRIPTION OF SURFACES—ANNUAL MAINTENANCE COSTS—ROUTE NO 12—LEOMINSTER TO CONNECTICUT LINE

27,558 Miles State Highways, 13,911 Miles Town and City Roads

Auto Route No	Maint Route No	Town	Maint Sec No	Length Miles	Year Constructed	Description of Existing Surface: Type	Width Feet	Area Sq Yds	Thickness Inches: Top	Thickness Inches: Base	Asphalt Penitra Gal Per s y	Foundation: Type	Thickness Inches	Shoulders: Type	Width Feet	Thickness Inches	Treatment	Annual Maintenance Cost—Per Mile: Surface	Right of Way	Operating: Snow Removal	Registration	Traffic Police	Eng and Adm 15% on (19) (20) (21)	Total Maint (19)-(24)	Total Annual Maint Per Sq Yd	Remarks
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)
12	4	Leominster	1	3 5	—	BMA	18–25	—										$—	$—	$—	$—	$—	$—	$—	$—	Town Road
		Leominster	2	0 371	1929	BMA	24	5,223	2½	4½	2¼	Gravel	12-Var †		1			5	600	239	195	134	127	1 300	0 092	† Variable
		Leominster	3	2 161	1927	BMA	24	30,427	2½	2	2¼	Stone	8					9	570	239	195	134	123	1 270	0 090	
												Gravel	4	Gravel												
		Sterling	1	0 592	1927	BMA	24	8,335	2½	2	2¼	Stone	8					9	570	235	195	134	122	1 265	0 090	
												Gravel	4	Gravel												
		Sterling	2	1 759	1913	BCA	18	18,575										194	458	177	195	134	124	1,282	0 121	
		Sterling	3	0 701	1913	WB	18	7,402										409	458	177	195	134	157	1,530	0 145	
		Sterling	4	0 3	—	BMA	15–30											—	—	—	—	—	—	—	—	Town Road
		Sterling	5	1 296	1913	WB	15	11,404										409	458	147	229	158	152	1,553	0 176	
		Sterling	6	1 087	1912	BCA	15	9,566										194	458	147	221	152	120	1,292	0 147	
		Sterling	7	0 093	1929	BMA	24	1,308	2½	4½	2¼	Gravel	Var					22	458	235	213	147	107	1 182	0 084	
		Sterling	8	0 042	—	BMA		—										—	—	—	—	—	—	—	—	Town Road
		Sterling	9	0 232	1929	BMA	24	3,266	2½	4½	2¼	Gravel	Var	Gravel	1	6		22	458	235	213	147	107	1 182	0 084	
		W Boylston	1	0 898	1928	BMA	24	12 644	2½	2	2¼	Stone	10					51	328	186	211	145	85	1 006	0 071	Sec 1 and 4 are
												Gravel	4													Adjacent
		W Boylston	4	0 653	1915	BCA	18	6 897										354	328	140	205	141	123	1 291	0 122	
		W Boylston	5	0 736	1927	BMA	30	12,954	2½	4½†	2½	Gravel	6§	Gravel				51	328	233	323	222	92	1,249	0 071	§ Only under 12 ft Widened Portion
		W Boylston	6	0 017	—	BCA		—										—	—	—	—	—	—	—	—	Town-Bridge
		W Boylston	7	0 459	1925	BMA	18	4,847				Gravel	12 Var					51	328	140	323	222	78	1,142	0 108	

		W Boylston	8	1 672	1932	BMA	30	29 427	2½	4½	2½							30*	300*	300*	323	222	94	1 269	0 072	* Estimated
		Worcester	1	1 074	1932	BMA	30	18, 902	2½	4½	2½	Gravel	Var					30*	300*	300*	323	222	94	1 269	0 072	
		Worcester	2-6	7 4	—	Miscl	—											—	—	—	—	—	—	—	—	City Streets
12	4	Auburn	1	1 634	1918	BMTR*	21	20, 130										988	393	223	517	356	241	2 718	0 221	* Retread surface Laid by Maint Forces in 1930
		Auburn	2	0 026	—	Plank		—										—	—	—	—	—	—	—	—	R R Bridge
		Auburn	3	0 258	1918	BMTR*	21	3 178										988	393	223	517	356	241	2 718	0 221	
		Auburn	4	1 430	1924	RCC	20	16 778	8			Gravel	6-12	Gravel	2	8	2½ gal	119	393	223	517	356	110	1 718	0 146	
		Auburn		1 361	1925	RCC	20	15 969	8			Gravel	6-12	Gravel	2	8	2½ gal	119	393	223	517	356	110	1 718	0 146	
12		Auburn		0 570	1925	RCC	40	13 376	8									150	393	446	830	572	148	2 539	0 108	Widened from 20 to 40 ft in 1932
20					1932																					
12		Oxford	1	0 894	1925	RCC	20	10 489	8			Gravel	12 Var	Gravel	3	8		88	299	223	249	171	91	1 121	0 096	
		Oxford	2	3 164	1921	PCC	20	37 123	6			Gravel	6-9	Gravel	4	4		112	299	223	262	181	95	1 172	0 100	
												Stone	6-9	Gravel	2	4										
		Oxford	3	0 9	1920	BMT	18	—										—	—	—	—	—	—	—	—	Town Road
		Oxford	4	2 290	1913-16	BMT	15	20, 152										404	299	167	319	220	130	1 539	0 175	
		Webster	1	0 831	1911	BMT	18	8, 775										319	183	134	418	288	95	1 437	0 136	
		Webster	2-6	1 25	—	BMT	18	—										—	—	—	—	—	—	—	—	Town Road
						PCC																				
		Dudley	1-2	0 46	1920	PCC	18-30	—										—	—	—	—	—	—	—	—	Town Road
		Dudley	3	0 464	1920	PCC	18	4, 900	7½ Center 5 Sides			Gravel	6					216	183	196	232	159	89	1 075	0 103	
		Dudley	4	0 016				—										—	—	—	—	—	—	—	—	R R Grade Crossing
		Dudley	5	0 878	1920	PCC	18	9 272	7½ Center 5 Sides			Gravel	6					216	183	196	232	159	89	1,075	0 102	

↑ Connecticut Line

Pavement Legend BMA—Bituminous Macadam—Asphalt BMT—Bituminous Macadam—Tar BCA—Bituminous Concrete—Asphalt BCD—Bituminous Concrete—Cold Type PCC—Plain Cement Concrete RCC—Reinforced Cement Concrete Dual—Two lanes of RCC divided by one lane of BMA WB—Waterbound Macadam (Surface Treated) GR—Gravel (Surface Treated) may have two or more auto route numbers But the Maintenance Route Numbers usually remain unchanged from year to year, and there are no duplications For convenience road cost data were assembled by Maintenance Routes, but the equivalent auto route numbers are given to identify the routes as known locally or as shown on road maps Throughout this report the route shown in Table I will be referred to as State Highway Route 12, because this is the number by which it is best known

MAINTENANCE COST DATA

Surface and Right-of-Way Maintenance costs shown in columns (19) and (20), Table I, are in most cases the average costs for years 1928–32, inclusive For roads less than five years old, an average of the maintenance costs was taken for the number of years available, and for new pavements where no records were available the annual maintenance cost was estimated based on records of other similar pavements Average costs over five years were used rather than actual costs for a single year, because surface maintenance costs often vary from year to year particularly for a road requiring a seal coat every two or three years

Costs were taken directly from the maintenance records of the Third District, which includes all of Worcester County These records separate maintenance costs as follows.

Surface Repairs
Surface Treatment
Roadbed
Drainage
Right-of-Way
Traffic Markings
Trees

Of the above classification "surface repairs" and "surface treatment" have been combined in column (19), Table I, and called "surface," and all the other items have been combined in column (20), Table I, and called "right-of-way "

In Massachusetts, surface items include only work done on the paved portion of the road Repairs to shoulders and ditches are included in "roadbed " This may explain why surface costs in Massachusetts are low and right-of-way costs are high in comparison with other states

Surface costs are segregated by types as indicated by section numbers in column (4), Table I Right-of-way costs are not separated by types of pavement, but by township boundaries Hence, for the state highway portions of any route lying within a certain town there will be only one cost figure available for right-of-way maintenance, but there may be several cost figures available for surface maintenance depending upon the number of pavement sections in that town Wherever right-of-way costs were found separated between sections in the same town, advantage was taken of the separation, and the right-of-way maintenance will not necessarily always be the same within each town

Marking of traffic lanes belongs strictly in operating maintenance This work is done, however, by the ordinary maintenance crew and is not charged to the account of the Traffic Engineer and his staff For this reason, and also to avoid the work of separating this item for every section, it was included with right-of-way maintenance

OPERATING MAINTENANCE

Snow Removal costs were obtained by averaging the costs for the two winters 1930–31 and 1931–32 The average snowfall for these two years was nearly the same as the average over many years, and therefore the average cost for these two years was considered as representative of a typical year

Snow removal costs were obtained from District Engineer's reports which segregate costs by auto routes, and also for each town through which the route passes No separation is made in original records between sections of different type or width of pavement In order to take into account the different widths of pavement between town boundaries on the same route, the average cost of snow removal per mile for the entire length of route ploughed within the town was allocated to the several sections in proportion to their width The costs thus obtained are shown in column (21) Table I

Registration Expense The expenses of operating the registry of motor vehicles in 1932 amounted to $1,508,236, which has been allocated to each mile of road in the ratio of vehicle miles traveled on that road to vehicle miles traveled in the entire state, including all rural roads and city streets The latter was estimated at 6,700,000,000 vehicle miles, based on a total gasoline consumption for all classes of vehicles of 560,194,000 gallons in 1932, at an average rate of one gallon consumed for every 12 miles of travel

This method of distribution spreads the registration costs over all roads, state, local, and city, in proportion to their use by motor vehicles, and results in much higher charges per mile to heavily traveled roads than to the lighter traveled roads The registry charges made against road sections on Route 12 are shown in Table I, Column 22

The method of distributing the registration expense back to the roads was given considerable thought Two other methods were considered and abandoned as less logical than that adopted The abandoned methods were

(1) Deduct registration expense from registration and license fee receipts and then allocate what is left This method was abandoned because it leaves out of consideration an important annual cost which is paid out of the motor vehicles tax contributions If a complete

picture is to be gained of the relation between highway income on the one hand and annual road cost on the other, all items of expense should be included

(2) Consider the registration expense as a stand-by "ready to serve" charge for all public ways anywhere in the State and levy the charge as a flat rate to each mile of road There are approximately 24,600 miles of public roads, including state, local and city highways Dividing the expense of the registry, $1,508,236 by 24,600 miles gives an annual flat rate cost of $61 30 per mile of public road This method was abandoned because it makes the same charge against a dirt road on a country by-way as for a heavily traveled state road or city street

Traffic Control The expenses of the Traffic Division in the Department of Public Works for 1932 were $189,144, which has been allocated to each section of road in the ratio of vehicle miles traveled on that section to the total vehicle miles traveled in the state, i e , in the same manner as registration expense was distributed The traffic control costs were charged against all roads rather than only to state highways, because although the Traffic Division is a state organization, it also takes counts on town roads, gives advice to towns and cities on traffic control problems, and must by law approve all legal traffic control signs and signal control installations located anywhere in the state

Policing The policing of state highways is performed by state police operating under the Department of Public Safety, which is not a part of the Department of Public Works These police devote part of their attention to the regulation of traffic and law observance among automobile drivers, but their duties also extend to the protection of the public anywhere along their route For this reason only a part of their cost should be charged as a road cost In 1932 the amount expended for state police from motor vehicle contributions was $312,028, which is roughly one-third of the cost of maintaining these police This amount has been allocated to the state highway system in the ratio of vehicle miles traveled on any particular section of road to the total vehicle miles traveled on state highways The latter has been estimated at 2,450,000,000 vehicle miles per year, which is equivalent to an average traffic of 3700 vehicles per day on every mile of the 1800 miles of state highways The estimate of vehicle miles on state system was obtained from actual counts, not from gasoline consumption It is interesting to note that in Massachusetts 36 5 per cent of the total vehicle miles traveled is on state highways In the Michigan traffic survey published in "Public Roads," February, 1933, it was found that about 33 per cent of all travel was on the state highway system of that state . The Michigan state system, however, has 7691 miles, and the average traffic per day on this system is only 1143 vehicles per day, compared with 3700 on the Massachusetts state system

ENGINEERING, SUPERVISION, AND ADMINISTRATION ON MAINTENANCE

In Massachusetts these costs average about 15 per cent of the total of surface maintenance, right-of-way maintenance and snow removal costs The percentage was not applied to registration, traffic, or policing costs because these are not supervised by the maintenance staff The percentage method of obtaining these costs is not recommended if actual administrative costs of each section are known The figure, 15 per cent, was chosen as a result of a study of several recent annual reports of the Department of Public Works and represents the ratio between maintenance expenses and all maintenance overhead items for the entire state system It appears high when compared with 0 43 per cent and 5 5 per cent used in cost analyses of Ames-Des Moines, Iowa, road, and Boston Post Road, Conn , respectively (See 10th Annual Report Highway Research Board) However, the 15 per cent includes many items not included in the former analyses It not only includes the actual supervision of road repairs but also the engineering and overhead expenses at district offices and at the main office in Boston

CONSTRUCTION HISTORY AND CAPITAL COST

Prior Construction—Its Value to Present Roads

The value of prior construction to an existing road must be estimated as it forms part of the capital value In Massachusetts the state highway system has been built up gradually over a period of nearly 40 years, with the result that nearly all of the existing state roads have had one or more prior constructions It was necessary therefore to investigate the construction history of each section of pavement in order to arrive at a reasonable figure for the value of this prior construction to the present road structure This value could have been obtained from records of old contracts together with the layout plans which show the extent to which the old road was utilized in the present construction, but for a great many sections, such as are included in the Worcester study, this method would have taken more time than its importance justified It was therefore decided to limit the investigation of prior costs to the data available in Annual Reports of the Massachusetts Highway Commission Fortunately these reports included down to 1921 an accumulative table of construction expenditures by sections of state highway which could be identified with maintenance sections used in this report The figures in column (8), Table II, were obtained from this source

These figures include only the costs for the first construction performed by the state on any road, which usually occurred directly after the road was taken into the state highway system Some of these roads, particularly the older ones, have been resurfaced one or more times between their first construction and the present construction

TABLE II

Construction History—Capital Cost—Route No 12—Leominster to Connecticut Line—State Highway Sections only

Auto Route No	Maint Route No	Town	Maint Sec No	Length, Miles	Prior Construction: Year Built	Prior Construction: Type of Surface	Prior Construction, Costs Per Mile: Original Const	Prior Construction, Costs Per Mile: Salvage Value at 30% of (8)	Prior Construction, Costs Per Mile: (9) Brought to Av 1928–32 Prices	Prior Construction, Costs Per Mile: Value to Present Rd Adopted	Present Road: Year Built	Present Road: Type of Surface	Present Road, Costs Per Mile: Construction Cost	Present Road, Costs Per Mile: Engineering and Over Head at 13% of (14)	Present Road, Costs Per Mile: Cons Cost + Engr & Over Head	Present Road, Costs Per Mile: (16) Brought to Av 1928–32 Prices	Present Road, Costs Per Mile: Right of Way	Present Road, Costs Per Mile: Betterments	Present Road, Costs Per Mile: Total (17) + (18) + (19)	Total Capital Cost Per Mile [(11) +(20)]	Remarks
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	
12	4	Leominster	2	0 371	1915	BCA	$21,432	$6,430	$7 610	$7 600	1929	BMA	$26,525	$3,448	$29,973	$24,853	$731	$ —	$25 584	$33 184	Resurfaced and widened in 1929
		Leominster	3	2 161	1901–2	GR	9 876	2,963	4 165	4,100	1927	BMA	48,784	6,342	55,126	39 616	731	—	40 347	44 447	Reconstructed, 1927
		Sterling	1	0 592	1913	BCA	9 940	2,982	3,089	3 000	1927	BMA	48,784	6 342	55 126	39 616	731	336	40 683	43,683	Reconstructed, 1927
		Sterling	2	1 759						None	1913	BCA	9 940	1,292	11 232	11 633	731	336	12 700	12 700	
		Sterling	3–5	1 997						None	1913	WB	8 825	1,147	9 972	10 328	731	—	11,059	11 059	
		Sterling	6	1 087						None	1912	BCA	8 359	1 087	9 446	10 417	731		11 148	11 148	
		Sterling	7–9	0 325	1912	BCA	8 359	2 508	2 765	2 800	1929	BMA	37,577	4 885	42 462	35,208	731	—	35 939	38 739	Reconstructed, 1929
		W Boylston	1	0 898	1913	WB	15 700	4,710	4,878	4 900	1928	BMA	29,555	3 842	33 397	24 609	731	—	25 340	30,240	Reconstructed, 1928
		W Boylston	4	0 653						None	1915	BCA	14,098	1,833	15 931	18 853	731	—	19 584	19 584	
		W Boylston	5	0 736	1915	BCA	14,098	4,229	5 005	5,000	1927	BMA	28,553	3,712	32,265	23 187	731	—	23 918	28 918	Resurfaced and widened in 1927
		W Boylston	7	0 459	1915	BCA	14,098	4 229	5,005	5 000	1925	BMA	48 912	6,358	55 270	33 056	731	—	33,787	38 787	Reconstructed, 1925
		W Boylston	8	1 672						None	1932	BMA	30 052	3,907	33 959	56 892	731	—	57 623	57 623	New location
		Worcester	1	1 074						None	1932	BMA	30,052	3,907	33 959	56 892	731	—	57,623	57 623	New location
		Auburn	1–3	1 892						5 800	1918	BMT*	16 274	2 116	18 390	10 206	731	108	11 045	16 845	* Retread Surf added in 1930, charged to Maint
		Auburn	4	1 430	1898–					5 800	1924	RCC	69 394	9 021	78 415	45 149	731	—	45 880	51 680	
		Auburn	4	1 361	1903	WB	13,200	3,960	5,761	5 800	1925	RCC	59 165	7 691	66 856	42 667	731	—	43 398	49 198	
		Auburn	4	0 570						5,800	1925	RCC	59 165	7 691	66,856	42,667	731	—	43 398	49 198	
12 and 20										None	and 1932	RCC	22 379	2 909	25 288	41 839	—	—	41 839	41 839	Widening to 1925 pavement
12		Oxford	1	0 894	1906–7	WB	10,350	3,105	3,710	3 700	1925	RCC	59 165	7 691	66 856	42 667	731	—	43 398	47 098	Reconstructed, 1925
		Oxford	2	3 164						None	1921	PCC	45 899	5 967	51 866	30 271	731	50	31 052	31 052	
		Oxford	4	2 290						None	1913–16	BMT	15 340	1 994	17 334	16 460	731	—	17,191	17 191	
		Webster	1	0 831						None	1911	BMT	12 257	1 593	13 850	15 174	731	—	15 905	15 905	
		Dudley	3–5	1 342						None	1920	PCC	48,643	6 324	54 967	24 825	731	—	25,556	25 556	

For Pavement Descriptions and Legend see Table I

The cost of these resurfacings has not been included in the cumulative tables in the Annual Report mentioned above, because this subsequent work is classified as reconstruction under the general heading of "Maintenance" in the state highway accounts Time has not permitted running down each of these resurfacing contracts to determine what added salvage value they may have contributed to the existing structure The salvage value allowed for the prior construction is, therefore, conservative This omission does not introduce any considerable error because the resurfacings referred to were in the nature of replacements of original road surface made during the period 1908–20 when few changes were made in location Reconstruction on a large scale, transforming old pavements into modern pavements, has been accomplished for the most part during the last 15 years, and these pavements are still in service, and are therefore carried at their full cost in column (14)

The proportion of the cost of prior construction carried forward to the present road was arbitrarily taken as 30 per cent of original cost regardless of the type of pavement produced by the prior construction The 30 per cent is intended to represent the portion of the original cost invested in the more permanent parts of the construction, namely, the grading and durable drainage structures No value has been assigned to the old surface material which may or may not be of value to present road, because usually much of the old surface is either abandoned, covered up, or excavated, in order to provide alignment and grades suitable for modern traffic In places where relocations have been made the old grading likewise has no value to the new road The sections where the old surface serves as a foundation tend to balance the sections where old locations have been abandoned, so that the 30 per cent allowance appears to be a reasonable percentage to use generally The relation between cost of grading and cost of pavement is not the same for all types of surface, but considering the uncertainties involved in estimating prior construction costs in the first place it did not seem advisable to adopt different percentages for different types The early bituminous types and the waterbound pavements show about the same ratio between surface and grading costs, the gravel roads have a much smaller proportion in the surface However, the grading and drainage structures provided for gravel roads, particularly the older ones, are generally of a lower order than those provided for higher type surfaces and for that reason usually of less value to surfaces which replace them For this reason the 30 per cent is not unreasonable when applied to these old gravel roads If the gravel roads had been constructed as a step in "stage" construction as is common today, then their grading and surface could and would be fully utilized in construction of a higher type pavement

The "Adopted Value to Present Road" in column (11), Table II, was obtained by rounding off the figures in column (10) Where the word "none" appears, this means that no previous pavement has been constructed by the state on that location, i e , the present road may be on new location, it may have replaced an undeveloped country byway, or it may have replaced a surfaced town road In the latter case the previous road probably was of some value to the present one; but usually these town roads required reconstruction by the state upon becoming a part of the state system

Capital Cost of Present Road

The construction cost for each section of present road was obtained from the final amount paid to the contractor at the conclusion of the contract under which the work was done This amount divided by the length of the project in miles gave the amounts recorded in column (14), Table II No attempt was made to separate construction costs between grading, drainage and pavement, as the final estimate of cost prepared by the state does not contain this separation The only way to obtain such a separation would be to make a quantity separation from the engineer's final estimate and apply bid prices to these quantities

Engineering and overhead on construction shown in column (15), Table II, was taken as 15 per cent of contract construction cost This percentage was obtained by examining the financial statements of Public Works Department for several years back It applies to state highway construction throughout the state, and is intended to include all overhead costs incident to construction both in the district offices and in the main office in Boston

Cost Index

In order to place the construction costs of the many sections of road on a comparable basis, it was necessary to bring the costs to some common price level In previous analyses presented by the Committee, the Engineering-News Cost Index was used, and costs brought down to the date of the analysis In the present study a new cost index has been developed based on contract prices in Massachusetts, which is therefore directly applicable to Massachusetts costs The common level chosen was an average of 1928–32 prices If the costs had been brought "to date," the date used would have been November 30, 1932, which is the end of the Massachusetts Department of Public Works' fiscal year As 1932 prices were at a low ebb in road construction history, costs brought to this level would be so low as not to be representative of either past or expected future conditions The average of 1928–32 prices reflects both a high and a low period and is therefore a more representative basis of comparison This also places construction costs on the same basis as maintenance costs which were averaged over the same period, 1928–32

The cost index curves developed are shown in Figure 2. They are based upon contract bid prices on state highway work as published annually in reports of Massachusetts Highway Commission (1895–1919) and Department of Public Works (1920–32). An unweighted average was taken for each year of the amounts bid on the following items:

Earth Excavation—as an index of grading costs
Plain Cement Concrete—as an index of drainage costs
Broken Stone—as an index of surface costs
Asphalt—as an index of surface costs
Gravel—as an index of foundation cost
Concrete Surfacing—as an index of concrete surface costs

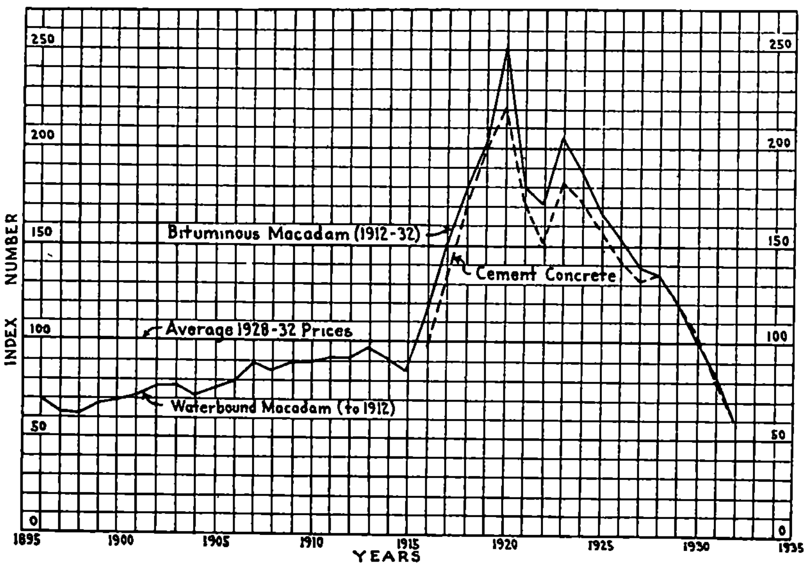


Figure 2. Massachusetts Highway Construction Cost Index

The number of contracts averaged per year varied from 40 to 160, depending upon number of contracts let.

The composite curve for bituminous macadam and waterbound roads (Figure 2) was obtained by weighting the above items approximately in proportion as they made up the cost of these road as follows:

	Bituminous Macadam 1912–1932 %	Waterbound Macadam 1895–1911 %
Grading (Earth Excavation)	30	30
Drainage (Plain Cement Concrete)	10	10
Foundation (Gravel Borrow)	10	10
Surface (Broken Stone)	40	50
Surface (Asphalt)	10	
	100	100

Few bituminous roads were built prior to 1912, so the asphalt item was not available until that date, however, the influence of this item on the index was so slight that the index curve shows no break between years 1911 and 1912

The index curve for bituminous macadam (asphalt) was also used for adjusting costs of bituminous concrete and tar macadam roads These three types are enough alike so that the index for one was considered applicable to all The same cost index, where it applies to waterbound roads, was also used for gravel roads There were so few miles of gravel road to be considered that a separate index for this type was not warranted Sufficient study was made of the cost trends of these two types to discover that they were nearly alike

A separate cost index was prepared for concrete roads, because it was found impracticable to combine concrete and broken stone roads into one index Concrete roads have only been built since 1916, and show different cost trends from the roads constructed of broken stone and bituminous materials The weighting used for concrete road cost index was as follows:

Grading (Earth Excavation)	20%
Drainage (Plain Cement Concrete)	10
Foundation (Gravel Borrow)	5
Surface (Concrete Surfacing)	65
	100%

The shape of the index curves depends not only upon the price fluctuation of the several classes of materials making up the indexes but also upon the percentage of the total cost assumed for each material During the period from 1923 to 1932 the unit prices bid on highway work in Massachusetts declined every year, but not at the same rate Excavation and gravel borrow dropped the most, from $1 48 and $1 80 per cu yd respectively in 1923 to $ 24 and $ 29 respectively in 1932 During the same period broken stone declined from $3 92 per ton to $1 66, and concrete surfacing declined from $12 99 per cu yd to $5 47 per cu yd The former pair of materials decreased in cost in the ratio of 6 to 1: the latter in the ratio of 2 4 to 1. Evidently the excavation and gravel borrow items tend to make the indexes decline sharply during the last ten years, whereas the other items tend to make the decline less marked The percentages chosen were based upon a study of cost separations given in several of the Annual Reports of the Department of Public Works, supplemented by estimates of cost separations for modern designs The Bureau of Public Roads index ("Public Roads," July, 1933) was also studied This index is based upon a "composite mile" of road which shows the following cost separation.

Excavation	35%
Surface (concrete + steel)	54%
Drainage (structural concrete + structural steel)	11%
	100%

The index developed in this report is not intended, however, to reflect the variation in cost of a mile of road of the same general type, but rather to indicate what a road constructed in any year would have cost if built of identical design at the chosen base price level (1928–1932) The actual cost of road construction per mile has not declined in anything like the manner shown by the index curves because as the width and depth of pavement have been increased for heavier vehicles more materials have been required per mile and more grading has been required to provide greater width of pavement, flatter curves and grades and wider shoulders The increase in usage of highway materials per

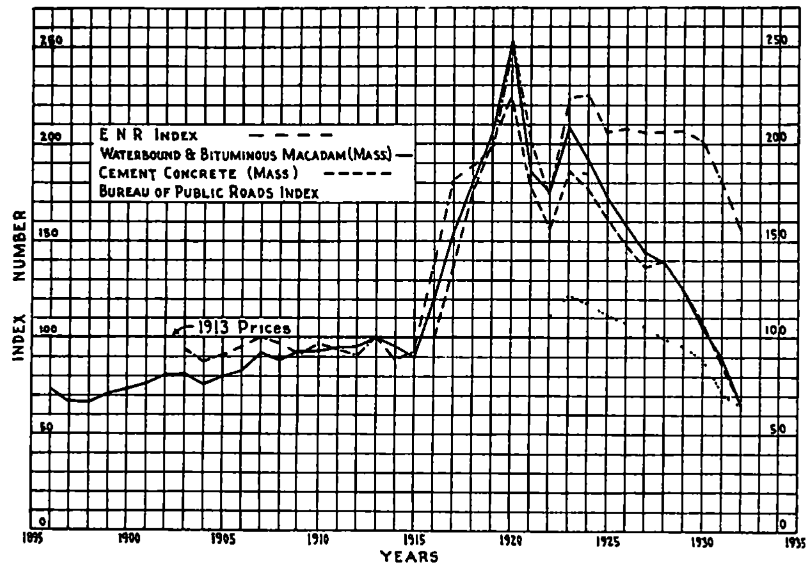


Figure 3 A Comparison of Engineering News-Record Building Construction Cost Index, Bureau of Public Roads Highway Construction Cost Index and Massachusetts Highway Construction Cost Index

mile is discussed at length in "Public Roads," July, 1933, referred to above

For purpose of comparison, the Massachusetts highway construction cost indexes, the E N R index, and the Bureau of Public Roads index have been plotted to the same base level in Figure 3 This figure shows at a glance that neither the E N R nor the Bureau of Public Roads indexes follow the treand of highway construction costs in Massachusetts between years 1922 and 1932 The E N R and the Bureau of Public Roads indexes are both intended to reflect the average national price trends, the former based on general or building construction material and labor, and the latter on highway construction materials with heaviest weighting given to those used in reinforced cement concrete type of construction The E N R index basis is such that it would not necessarily reflect highway trends, but one might expect the Public Roads curve and the Massachusetts curves to be quite similar An inspection of the basic data for the Public Roads index shows, however, that in 1922 Massachusetts highway prices far exceeded those for the nation as a whole For example, excavation in Massachusetts averaged $1 16 per cu yd for this year, whereas the average for the country was only $0 40, but in 1932 the national average and Massachusetts prices were almost alike Figure 3 definitely shows the unreliability of general or average cost indexes for application to the highway construction costs of a particular state

Right-of-way costs were obtained by arbitrarily assuming a cost of $100 per acre for a right-of-way 60 feet wide In most cases the state did not pay anything for right-of-way, but merely took over the existing one when the road became a state highway On new locations where land damages were paid, the actual cost of these will appear in column (18) Table II instead of the nominal figure of $731 per mile

Betterments

In recent years maintenance costs in Massachusetts have been kept in two accounts, ordinary maintenance and betterments Ordinary maintenance costs are those given in columns (19) and (20), Table I Betterments were considered capital expenditures and therefore included with road costs in Table II, column (19) The betterments were not brought to a common cost level, as they all fall within the period 1928–32, and are only a small item in the total road cost Betterments include such work as paving the space left by street railway rails, installing additional drainage structures, and widening of isolated curves

CALCULATION OF ANNUAL ROAD COSTS

Annual road costs were computed by an approximate formula as follows:

$$C = \left(\frac{A + S}{2}\right) r + \frac{A - S}{n} + B + \frac{E}{n},$$

in which

C = average annual road cost
A = original capital cost
B = annual maintenance cost
r = rate of interest
n = estimated life, in years, of the surface before renewal is required
S = estimated salvage value of highway at the end of n years
E = any periodic maintenance required during life n.

The term $\left(\frac{A + S}{2}\right) r$ is the approximate average annual interest charge during the life n, and the term $\frac{A - S}{n}$ is the annual depreciation on a straight line basis

The formula differs in principle from the basic formula for road cost presented at 1929 meeting of Highway Research Board, because it sets up road cost for the period n years only instead of in perpetuity It also neglects compound interest The approximate formula is simple in form and practical in its application, because it does not call for any assumptions beyond the life of the existing surface The method of amortizing capital invested is similar to that used for retiring serial bonds The most difficult terms to evaluate are n and S

The results obtained by using the approximate formula agree closely with those found by the exact one as brought out by the following illustration

The exact formula may be written as follows

$$C = Ar + B + \frac{Er}{(1 + r)^n - 1} + \frac{E'r}{(1 + r)^{n'} - 1}$$

where

C = Annual Road Cost
A = Cost to construct (capital cost)
B = Annual maintenance cost
r = Rate of interest (4 per cent)
n = Life, in years
E = Replacement Cost at end of n years
E' = Periodic Maintenance needed every n' years

Assuming the following costs A = \$30,000, B = \$1000, r = 04, n = 20 years, E = \$20,000, E' = 0

Then

$$C = 30{,}000 \times 04 + 1000 + \frac{20{,}000 \times 04}{1\ 191} + 0 = \$2871$$

Using the Approximate Formula, where $S = A - E = \$10{,}000$,

$$C = \left(\frac{A + S}{2}\right) r + \frac{A - S}{n} + B + \frac{E'}{n}$$

$$C = \left(\frac{30{,}000 + 10{,}000}{2}\right) 04 + \frac{30{,}000 - 10{,}000}{20} + 1000 + 0 = \$2800$$

Estimated Life of Present Surface

The lengths of life estimated for the different pavements on Route 12, Table III, are indicated in column (9) of the table The bituminous macadam roads (BMA) have been given a life of 20 years This type as now built was developed about 1923 no roads of this design are old enough to have worn out Roads of this type constitute 65 per cent of the Massachusetts state highway system They have the lowest surface maintenance cost per square yard of any type except the dual type in which two lanes of concrete are separated by one of bituminous macadam There are some miles of an older type of bituminous macadam that are now 20 years old and still giving good service Judging by the performance of these roads, 20 years seems a proper estimate of the life of the "high type" bituminous macadam pavement before any new surface layer must be added Allowance has been made for one seal coat during the 20-year life This is item E in column (15), which has been spread uniformly over the 20-year period by dividing by n Bituminous macadam roads penetrated with asphalt do not require a seal coat as often as do roads penetrated with tar Massachusetts engineers expect many of these asphalt-bound roads to last longer than 10 years without a seal coat

The old pavements, sections 2, 3, 5, 6, in Sterling, section 4 in West Boylston, and sections 1, 3, in Auburn, are scheduled for reconstruction in 1934, each of these has been assigned life to 1934 Section 4 in Oxford and section 1 in Webster were being reconstructed in 1933, so they have been given their actual length of life

Reinforced concrete pavements (RCC) built since 1923 have been given life of 25 years before resurfacing with bituminous concrete or other materials will be needed These pavements are of 8-inch uniform thickness and are reinforced with about 100 lbs of steel per 100 sq ft

Plain concrete pavements have been given a life of 20 years, but provision has been made for covering these pavements with a layer of bituminous concrete at the end of 10 years It is evident from descriptions that the plain concrete sections in Oxford and Dudley have not been covered although they are now 12 and 13 years old, respectively. However, as they are pavements of inferior design, being only 5 inches thick at edges, they are badly cracked and will need resurfacing before long As the cost of this resurfacing (E) is distributed over the 20-year life (n), it makes no difference in the road cost whether the resurfacing is done at the end of ten years or later A "Symposium on Resurfacing of Pavements," published in the 12th Annual Proceedings of Highway Research Board, indicated that plain concrete pavements on heavily traveled routes require a surface layer when 10–12 years old Some plain concrete sections on Newburyport Turnpike in Massachusetts, of similar design to Oxford and Dudley sections, were covered at ten years The 20-year life assigned to plain concrete pavements represents two ten-year periods, one bare and one covered

The widening to the reinforced concrete pavement, section 4 in Auburn, represents a special case The 1925 and 1932 sections now constitute one pavement, so that when any surface layer is applied it will be applied to both at the same time, therefore, the 1932 sections (10 feet on each side) have been given a life of only 18 years, so that they will reach their salvage value in the same year as the 1925 pavement

Salvage Value of Present Surfaces

The salvage value of a highway at the end of the estimated life of the wearing surface is usually measured by the value of the grading, structures and surface left in that road as a foundation for a new wearing course If it is practical and desirable to place this new layer directly upon the old pavement, then the salvage value of the old pavement will be its original cost minus the cost of the new layer which is required to produce a road adequate for present traffic

Applying this principle to the high type of bituminous macadam so common in Massachusetts, the new surface layer required would probably be a 2-inch course of penetration macadam costing $8000 to $10,000 per mile for a 24-foot width at 1928–32 price level If the original pavement cost $30,000 per mile, then the salvage value would be roughly 70 per cent of original cost However, the intangible item of obsolescence should also be taken into consideration Many roads reconstructed during recent years have been relocated to obtain straighter alignment and flatter grades Wherever the new road did not follow the old, obviously the old road had no salvage value to contribute to the new road Most roads of recent design appear adequate to meet traffic demands for years to come, but there is no certainty that this will prove true

In highway financing it is not customary to issue refunding bonds against the residual value remaining indefinitely in the road, such as is represented by the "refunded debt" in railroad financing In fact, if money is borrowed at all, the bond issue is usually amortized in 10 years, a period less than the life of the pavement alone Massachusetts state highways are practically free from debt, for many years highway funds have been obtained exclusively from motor vehicle revenue

In view of these considerations a salvage value for the existing pavements of 30 per cent of capital cost has been assumed for all bituminous pavements including waterbound macadam with a surface treatment, and 40 per cent has been allowed for cement concrete pavements A higher value has been given to concrete because concrete pavements are usually laid with more attention to alignment and grades than are other types, so the likelihood of future relocation for better alignment is less

The percentage method of obtaining salvage value is not recommended where other facts are available which definitely influence the amount of salvage value left in a road The amount of salvage value adopted, however, can vary through a wide range without greatly influencing the total road cost, as illustrated by the following.

In the formula

$$C = \left(\frac{A + S}{2}\right) r + \frac{A - S}{n} + B + \frac{E}{n}$$

TA

SUMMARY SHEET—ROAD COSTS—TRAFFIC—VEHICLE CONTRIBUTIONS—ROU

Auto Route No	Maint Route No	Town	Maint Sec No	Type of Surface	Year Constructed	Length Miles	Width of Pavement Ft	Estimated Life (n) Yrs	Capital Cost A	Salvage Value S	Annual Road Cost—			
											Interest at 4% on $\frac{A+S}{2}$	Depreciation $\frac{A-S}{n}$	Annual Maint B	Periodic Maint E
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
12	4	Leominster	2	BMA	1929	0 371	24	20	$33,184	$9 955	$863	$1,161	$1,300	$70
			3	BMA	1927	2 161	24	20	44,447	13,334	1,156	1,556	1,270	70
		Sterling	1	BMA	1927	0 592	24	20	43 683	13 105	1 136	1,529	1,265	70
			2	BCA	1913	1 759	18	20	12,700	3,810	330	445	1,282	—
			3	WB	1913	0 701	18	20	11,059	3,318	288	387	1,530	—
			5	WB	1913	1 296	15	20	11,059	3 318	288	387	1,553	—
			6	BCA	1912	1 087	15	21	11,148	3,344	290	372	1,292	—
			7-9	BMA	1929	0 325	24	20	38,739	11,622	1 007	1 356	1,182	70
		W Boylston	1	BMA	1928	0 898	24	20	30 240	9,072	786	1,058	1 006	70
			4	BCA	1915	0 653	18	19	19,584	5,875	509	721	1,291	—
			5	BMA	1927	0 736	30	20	28,918	8,675	752	1,012	1,249	88
			7	BMA	1925	0 459	18	20	38,787	11,636	1,008	1 357	1,142	53
			8	BMA	1932	1 672	30	20	57,613	17 284	1,498	2,016	1,269	88
		Worcester	1	BMA	1932	1 074	30	20	57,613	17,284	1,498	2,016	1,269	88
		Auburn	1-3	BMT	1918	1 892	21	16	16,845	5,053	438	737	2,718	—
			4	RCC	1924	1 430	20	25	51,680	2,672	1 447	1 240	1 718	—
				RCC	1925	1 361	20	25	49 198	19,679	1,378	1 181	1,718	—
12 and 20				RCC	1925	0 570	20	25	49,198	19,679	1,378	1,181	2,539	
				RCC Widening	and 1932		20	18	41,839	16,736	1,171	1 395		—
12		Oxford	1	RCC	1925	0 894	20	25	47,098	18,839	1 319	1,130	1 121	—
			2	PCC	1921	3 164	20	20	31,052	12 421	869	931	1,172	587
			4	BMT	1913-16	2 290	15	20	17,191	5 157	447	601	1 539	—
		Webster	1	BMT	1911	0 831	18	22	15,905	4 771	413	506	1 437	—
		Dudley	3-5	PCC	1920	1 342	18	20	25 556	10 222	715	767	1 075	528

For Pavement Descriptions and Maintenance Costs see Table I
For Calculation of Capital Costs see Table II

assume A = $30,000, n = 20 years, B = $1000, E = 0, and r = 04
Then, if S = 60% of A,

$$C = \left(\frac{30{,}000 + 18{,}000}{2}\right) 04 + \frac{30{,}000 - 18{,}000}{20} + 1000 = \$2560$$

and if S = 30% of A,

$$C = \left(\frac{30{,}000 + 9000}{2}\right) .04 + \frac{30{,}000 - 9000}{20} + 1000 = \$2830$$

and if $S = 0$,

$$C = \left(\frac{30{,}000}{2}\right) 04 + \frac{30{,}000}{20} + 1000 = \$3100$$

Periodic Maintenance. A value for E was allowed for only two types of pavement For the bituminous macadam-asphalt (BMA) provision has been made for a seal coat costing $ 10 per sq. yd and applied at

III

12—LEOMINSTER TO CONNECTICUT LINE—STATE HIGHWAY SECTIONS ONLY

		Annual Traffic				Annual Road Costs			Vehicle Contributions—1932						
									Per Mile						
	Total Per Foot of Width	Passenger Cars	Light Trucks Up to 1½ Tons	Heavy Trucks 1½ Tons and over	Total (18) – (20)	Per Vehicle Mile	Per Ton Mile	Per Ton-Mile Per Ft of Width	Passengers Cars	Light Trucks	Heavy Trucks	Total (25)–(27)	Per Vehicle Mile	Per Ton Mile	Per Ton-Mile Per Ft of Width
	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)
4	141	738,000	57,000	71,000	866,000	$0 0039	$0 0023	$0 000096	$2,199	$231	$483	$2,913	$0 0034	$0 0020	$0 000083
2	169	738,000	57,000	71,000	866,000	0 0047	0 0028	0 000117	2,199	231	483	2 913	0 0034	0 0020	0 000083
0	167	738 000	57 000	71,000	866 000	0 0046	0 0027	0 000112	2,199	231	483	2 913	0 0034	0 0020	0 000083
7	114	738,000	57,000	71,000	866 000	0 0024	0 0014	0 000078	2,199	231	483	2 913	0 0034	0 0020	0 000111
5	122	738,000	57 000	71 000	866,000	0 0025	0 0015	0 000083	2,199	231	483	2 913	0 0034	0 0020	0 000111
8	148	872,000	66 000	81,000	1,019,000	0 0022	0 0013	0 000087	2,599	268	551	3 418	0 0034	0 0020	0 000133
4	130	840,000	63 000	78,000	981,000	0 0020	0 0012	0 000080	2,503	256	530	3,289	0 0034	0 0020	0 000133
5	151	812,000	60,000	76,000	948 000	0 0038	0 0023	0 000096	2,420	244	517	3 181	0 0034	0 0020	0 000083
0	122	803,000	59,000	75,000	937,000	0 0031	0 0019	0 000079	2,393	240	510	3,143	0 0034	0 0020	0 000083
1	140	781,000	57,000	72,000	910,000	0 0028	0 0017	0 000094	2,327	231	490	3,048	0 0033	0 0020	0 000111
1	103	1,186,000	61,000	188,000	1,435,000	0 0022	0 0012	0 000040	3,534	248	1,278	5,060	0 0035	0 0019	0 000063
0	198	1,186,000	61,000	188,000	1,435,000	0 0025	0 0013	0 000072	3,534	248	1,278	5 060	0 0035	0 0019	0 000105
1	162	1,186,000	61,000	188,000	1,435,000	0 0034	0 0018	0 000060	3 534	248	1,278	5,060	0 0035	0 0019	0 000063
1	162	1,186,000	61,000	188 000	1,435,000	0 0034	0 0018	0 000060	3 534	248	1,278	5 060	0 0035	0 0019	0 000063
3	185	1,922,000	75,000	299 000	2,296 000	0 0017	0 0009	0 000048	5 728	305	2,033	8 066	0 0035	0 0019	0 000091
5	220	1,922,000	75 000	299,000	2,296 000	0 0019	0 0010	0 000050	5 728	305	2,033	8,066	0 0035	0 0019	0 000095
7	214	1,922,000	75,000	299,000	2 296,000	0 0019	0 0010	0 000050	5,728	305	2 033	8 066	0 0035	0 0019	0 000095
4	192	3,157,000	257 000	274 000	3,688 000	0 0021	0 0012	0 000030	9 408	1,043	1 863	12,314	0 0033	0 0020	0 000100
0	178	945,000	66,000	94,000	1,105,000	0 0032	0 0019	0 000095	2 816	268	639	3,723	0 0034	0 0020	0 000100
9	178	996,000	80,000	90,000	1,166 000	0 0030	0 0018	0 000090	2 968	325	612	3,905	0 0033	0 0020	0 000100
7	172	1,210,000	109,000	99,000	1,418 000	0 0018	0 0011	0 000073	3,606	443	673	4,722	0 0033	0,0020	0 000133
6	131	1 594,000	110 000	153 000	1 857,000	0 0013	0 0008	0 000044	4,750	447	1 040	6 237	0 0034	0 0020	0 000111
5	171	883 000	60 000	86 000	1 029,000	0 0030	0 0018	0 000100	2,631	244	585	3 460	0 0034	0 0020	0 000111

the end of 10 years, the midpoint in the estimated life of surface. For plain concrete (PCC) allowance has been made for a surface wearing course costing $1 00 per sq yd, also presumably laid at the end of first 10 years The other types require bituminous surface treatments, but these are applied frequently and are included in annual maintenance cost

TRAFFIC

Traffic data were obtained from a state-wide census taken in August, 1933, by the Massachusetts Department of Public Works. The August counts were expanded to yearly volume on a basis of gasoline consumption The distinction between light and heavy trucks cannot be definitely drawn because this separtion as found in the traffic counts depended largely upon the counter's judgment His instructions were to count Fords and delivery trucks as light trucks, and all others as heavy trucks No weight or capacity limits were specified The light truck classification, column (19), Table III, includes, therefore, only trucks weighing little more than a passenger car The heavy truck classification, column (20), includes all other trucks and buses The buses were counted separately in traffic census, but have not been tabulated in this report

MOTOR VEHICLE CONTRIBUTIONS

(a) *State Taxes Collected*

Motor vehicle taxes collected by the State of Massachusetts for the fiscal year 1932 were as follows

Registration Fees, Drivers' License Fees and Examinations, and Court Fines	$ 6,337,418
Gasoline Tax	16,651,868
Total	$22,989,286

The above amounts represent the total contributions made by all classes of vehicles for the use of the public highways anywhere and everywhere in the state These taxes are paid into a special state account called "The Highway Fund" from which disbursements are made for the following purposes

Maintenance, construction and operation of state highways
Special projects authorized by legislative acts
State aid on town road construction and maintenance
Maintenance of the Metropolitan District Commission (for park roads and reservations in and around Boston)
Distribution of a portion of the gasoline tax receipts to the cities and towns

Of the above items the first and part of the second are for state highways Projects of unusual magnitude, such as the Boston-Worcester Turnpike, are authorized by special act and later become part of the state highway system Other projects authorized by special acts, notably the construction of park roads in the Boston Metropolitan area, are turned over to the Metropolitan District Commission for maintenance and do not become a part of the state highway system Of the $23,000,000, roughly, received from motor vehicle taxes in 1932, about $14,000,000 were spent on state highways, and about $5,500,000 of the gas tax money were distributed to the cities and towns The balance of the receipts were distributed among the other items The amounts expended upon state highways or allotted to the towns and cities will vary from year to year depending upon what action is taken by the legislature In recent years there has been a strong tendency to decrease expenditures on state highways and increase the proportion of the gas tax moneys given to the towns and cities For example, in 1931 about $16,000,000 were spent on state highways and $2,500,000 of gas tax money given to the towns and cities

(b) *Contributions from Individual Roads*

In order to estimate the motor vehicle contributions from any individual section of road it is necessary to express these total contributions in dollars per vehicle mile The amount contributed from any section of road will be the product of the vehicle miles traveled on that road and the contribution per vehicle mile The gasoline tax contribution is proportional to the number of miles driven and may therefore be readily expressed in dollars per vehicle mile The other fees, however, are independent of mileage and can only be expressed on a vehicle mile basis by making certain assumptions as to annual mileage The contribution per vehicle mile for each of the three classes of vehicles, passenger cars, light trucks and heavy trucks, has been worked out in Table IV The method of compiling this table is described below

(c) *Explanation of Table IV*

Average gross weights of vehicles under item (1) were chosen as a matter of judgment based on the data that were available The total number of vehicles registered, and the total number of trucks and buses registered were available from the registery, but the division between light and heavy trucks had to be estimated This was done by choosing the percentage distribution under item (4) These percentages are such as to satisfy two conditions. first, that the number registered in each class times the average registration fee for that class equals the total registration receipts, and secondly, that registration receipts from the light trucks plus the heavy trucks equal the difference between the total registration receipts and passenger car receipts The values chosen for miles per gallon of gasoline are low compared with figures compiled by the Iowa State Experiment Station (Bulletin 106) However, Massachusetts traffic is characterized by short runs, dense traffic in congested areas, and a relatively high percentage of the heavier passenger cars and trucks All these factors tend to increase the rate of gasoline consumption and thereby decrease the miles that may be driven per gallon of gasoline Vehicle miles for each class were estimated on three presumptions (1) that total gas tax receipts were obtained from each class of vehicles in proportion to the consumption of that class, (2) that the total vehicle miles for all classes equal 6,700,000,000 as previously determined, (3) that light trucks travel an average of 10,000 miles per year The resulting distribution indicates that passenger cars at 14 miles per gallon of gasoline travel an average of 8,000 miles per year, and heavy trucks at 6 miles per gallon travel 11,000 miles per year, which is a reasonable annual mileage for each class in Massachusetts, considering the fact that the heavy truck classification includes practically all trucks except light delivery trucks as explained under "Traffic" heading

Total registration receipts were available for passenger cars and for commercial vehicles unclassified The distribution between light and heavy trucks was made on the basis of average fee paid in each case Drivers' licenses, examination fees and court fines, item (11), were distributed among all classes in proportion to numbers registered Gasoline tax receipts, item (10), were distributed among classes of vehicles in proportion to their rate of consumption

TABLE IV

BASIC DATA FOR COMPUTING MOTOR VEHICLE CONTRIBUTIONS

Item	Passenger Cars	Light Trucks	Heavy* Trucks	Totals
(1) Average Gross Weight (Tons)	1¼	2½	5½	—
(2) Average Registration Fee	$3 60	$7 50	$16 65	—
(3) Number Registered (1932)	694,459	47,313	60,137	801,909
(4) Per Cent of Total Registered	86 6	5 9	7 5	100
(5) Miles Per Gallon of Gasoline	14	10	6	
(6) Vehicle Miles Per Year (Millions)	5566	473	661	6,700
(7) Registration Receipts (1932)	$ 2,490,759	$ 354,847	$1,001,216	$ 3,846,822
(8) License Fees, Fines, etc. (1932)	$ 2,156,856	$ 146,945	$ 186,795	$ 2,490,596
(9) Total Fees (7) + (8)	$ 4,647,615	$ 501,792	$1,188,011	$ 6,337,418
(10) Gasoline Tax Receipts† (1932)	$11,928,000	$1,419,000	$3,305,000	$16,652,000
(11) Total Fees [(9)] Per Vehicle Mile	$ 00084	$ 00106	$ 00180	$ 00095
(12) Gas Tax [(10)] Per Vehicle Mile	$ 00214	$ 00300	$ 00500	$ 00248
(13) Total Contributions (11) + (12) Per Vehicle Mile	$ 00298	$ 00406	$ 00680	$ 00343

* Includes 3899 Busses

† State Gasoline Tax in 1932 was 3 cents per gallon

The contributions per vehicle mile were obtained by dividing tax receipts from each class of vehicle by the vehicle miles traveled by each class

The contributions per mile of road for each class of vehicle shown in columns 25 to 28 of Table III were obtained by multiplying contributions per vehicle mile for each class by the number of vehicles of that class using the road

(d) *Contributions Considered as Highway Income*

In this report the contributions obtained from each section of road have been considered as the gross income from that road and have been set up in comparison with the road costs as expenses in columns 16 and 28 in Table III. In previous road cost analyses, namely, the Des Moines, Iowa, road and the Concord-Harvard and Tyngsboro roads in Massachusetts (10th and 12th Annual Proceedings of Highway Research Board) the vehicle contributions were multiplied by a factor, roughly two-thirds, which was intended to make allowance for the fact that only two-thirds of the gross contributions were devoted to state highways. Although such a disposition may have been made of the motor vehicle funds in the aggregate, it should not have been applied to the single sections of state highway studied if a true comparison is to be drawn between the road cost and the road earnings. The earnings of a particular road section depend upon the use made of it measured in vehicle miles. As indicated above the actual disposition made of the total highway funds in Massachusetts will fluctuate from year to year depending upon the whims of the legislature. The earnings of the state highway system as a whole may be estimated roughly from the figures previously derived as follows:

Vehicle miles on state system	2,450,000,000
Contribution per vehicle mile of the average vehicle	$.00343
Earnings of state system, gross income from motor vehicle contributions	$8,400,000
Expended on state highways in 1932	$14,000,000

A comparison between the earnings of the highway system and the amount actually expended upon the system in any year is not necessarily significant. Much of the actual expenditure is capital expenditures in new construction and reconstruction which is needed to relieve congested routes and to convert old routes of obsolete design into modern highways. The figure which should be set up in comparison with the annual earnings of the state highway system is the annual road cost of the system. This can only be obtained by an extension of the analysis outlined above to the entire state system.

Any comparison which is drawn between road cost and contributions is not complete unless the highway service provided is taken into consideration. For example, old roads usually have a low annual cost because their interest and depreciation charges are low, but these roads are not giving the service that a more expensive, higher type pavement would provide. The old roads are usually narrow and crooked, and have a wavy surface and a high crown. They are hard-surfaced, durable pavements, but cannot be traveled with the same speed and comfort as afforded by more modern pavements. Furthermore, the cost of vehicle operation is undoubtedly higher on these older roads than on modern types. The increment in vehicle operating costs between the old and new types in Massachusetts, at least, would be a small amount per vehicle because even the older roads are hard-surfaced, year-round roads and the difference in operating costs between them and a smoother type would be due largely to the elimination of the wavy surface and high crown No actual tests have so far been conducted upon road surfaces typical of Massachusetts types to determine just what the increments in vehicle operating cost would be between types These increments would have to be known with more precision than is now available in order to have any significance when expanded to the densities of traffic commonly found on Massachusetts highways No attempt has been made in this report to combine road costs and vehicle costs into one cost, namely, the cost of transportation

DISCUSSION OF THE BASIC DATA IN TABLE I

The maintenance costs for certain pavments on Route No 12 are so exceptionally high or low as to require explanation For example, Sections 2 and 3 in Leominster and 1 in Sterling show unusually low surface maintenance costs at $5 and $9 per mile, respectively, and high right-of-way maintenance costs at $600 and $570 per mile, respectively Normal maintenance costs for these sections, based upon a study of all roads of this age and type in Worcester County, would be about $25 per mile for surface maintenance and $300 per mile for right-of-way maintenance

Sections 1 and 3 in Auburn show excessively high surface maintenance (nearly $1000 per mile) They were originally constructed in 1918 as bituminous macadam penetrated with tar In 1930, however, a special surface treatment was given to these sections which is classed as a "retread" in the maintenance records It consisted of an application of ¾-inch pea stone probably not more than one inch average thickness which was penetrated with tar, mixed on the road, and then smoothed out and rolled At the same time the width of road was increased from 18 to 21 feet by extending the new surface over shoulders of the old road The cost of this treatment and subsequent applications of tar in 1931 and 1932 are responsible for the high surface maintenance cost The traffic on these sections is over 2,000,000 vehicles per year, of which 16 per cent are trucks, so it is not surprising that this type of surface has required extensive maintenance These Auburn sections are scheduled for reconstruction in 1934

Sections 8 in West Boylston and 1 in Worcester were completed late in 1932, hence no actual maintenance records are available. the costs shown in columns 19 and 20 were therefore based upon the records of other roads of similar type These records showed that the average surface maintenance during the first five years for the bituminous macadam asphalt type was very nearly $1 00 per mile per foot of width, and that the right-of-way maintenance was roughly $300 per mile

It is characteristic of the modern pavement sections analyzed in Worcester County to have much greater annual right-of-way maintenance costs than surface maintenance costs The former item, therefore, makes up a larger portion of the annual road cost than does the surface maintenance, which has been reduced to a small amount by building a modern type pavement suitable to traffic demands

Many published road costs prepared for the purpose of comparing roads of different surface types, omit the right-of-way maintenance costs, on the assumption that these costs will be the same for any kind of surface Such costs are not true road costs, because regardless of whether the right-of-way maintenance is a constant amount or not, it should be included, as it often contributes a substantial amount to the annual road cost, as illustrated by the sections analyzed on Route No. 12

In the Twelfth Annual Proceedings of the Highway Research Board, page 54, Mr Paustian has allowed a range of only $25 to $50 for "maintenance of shoulders," which is apparently intended to cover all right-of-way costs This is much less than the average of $300 per mile for Massachusetts roads It is probably true, however, that in certain states, particularly in the flat, midwestern area, the right-of-way costs will be small in comparison with surface maintenance costs

DISCUSSION OF RESULTS

Table III shows in tabular form a summary of the analysis made of each of the road sections The annual road costs and the motor vehicle contributions have been expressed in several different units taking into consideration both singly and in combination the effect of width of roadway and volume of traffic expressed both in numbers of vehicles and in gross weight of vehicles Vehicle miles were converted into ton-miles by using the average gross weight for each class of vehicle given in item (1) of Table IV The unit which best represents the annual road costs in terms of the road service provided is the ton-mile cost per foot of width shown in column (24) of Table III A comparison made between annual road costs expressed in this unit shows the relative economic efficiency of the different road sections, such as is not evident when the comparison is made on a per mile basis only For example, Sec 4 in Auburn has the highest annual cost per mile ($7664), but it has the lowest per ton-mile per foot of width (0 003¢) This short section of road not only serves Route 4 but also U S. Route 20 which is on the trunk line from Boston to New York Evidently this section of road is giving the most service for each dollar of annual cost Furthermore, the contributions from the dense traffic using this section of road greatly exceed the annual cost of this 40-foot reinforced concrete pavement The widening of this pavement from 20 to 40 feet in 1932 was therefore clearly justified

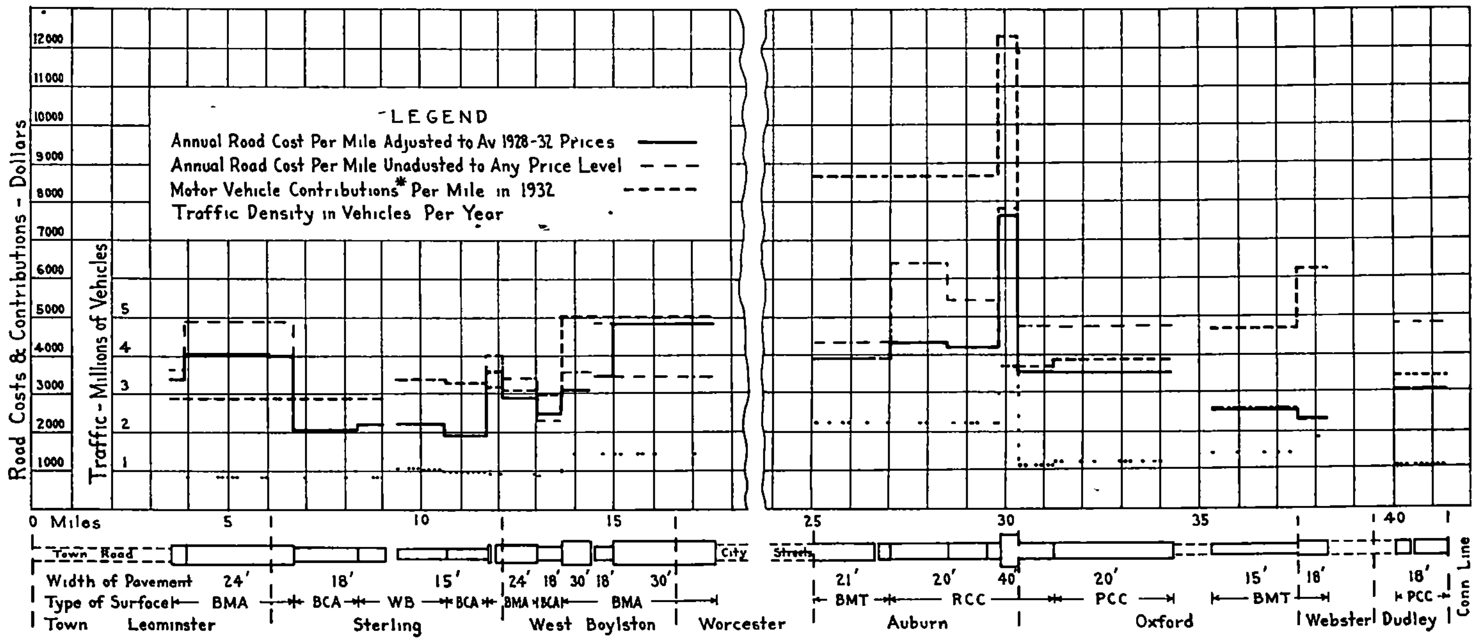
The sections giving the least service per dollar of annual cost are No 3 in Leominster and No 1 in Sterling, these cost 0 0117¢ and 0 112¢ per ton-mile per foot of width respectively An inspection of the motor vehicle contributions in column (28) reveals that these sections do not earn their annual costs

The motor vehicle contributions expressed in dollars per vehicle mile and per ton-mile in columns (29) and (30) are nearly alike for all sections These values would be exactly alike if the distribution of traffic between classes of vehicles agreed in every case with that assumed in Table IV, from which an average value for contributions per vehicle mile was derived, but as the actual traffic distribution varies somewhat between sections, the contributions per vehicle mile and per ton-mile will vary slightly

A better view of the results may be had from Figure 4 which shows graphically the annual road costs per mile plotted from column (16) of Table III, the motor vehicle contributions per mile for 1932 plotted from column (28), the annual traffic plotted from column (21), and another set of annual road costs which have not been adjusted to any base price level The calculations for these latter costs are not included in the report They were obtained by using costs from columns (9) and (16) in Table II in place of those in columns (11) and (17) of that Table The reasons for showing these unadjusted cost lines on the diagram were twofold· first, they indicate at a glance the extent to which the actual costs have been changed in adjusting them to the base price level, and, second, they make possible a comparison between the actual annual cost and the actual contributions As the actual costs are those which must be paid, this comparison has more significance for tax purposes than that between adjusted costs and actual contribution When comparing the road costs of sections constructed at different times, however, the adjusted costs should be used, because these eliminate cost fluctuations due to changing price levels Figure 4 shows that at the present rate of state motor vehicle fees and a 3-cent state gasoline tax the contributions exceed the annual road costs for all sections except Nos 2 and 3 in Leominster and Nos 1, 7 and 9 in Sterling

By taking into consideration not only Route No 12 but also other routes in the county for which costs have been compiled but not presented in this partial report, the following general conclusions may be drawn from Worcester County state roads

(1) Modern, two-lane roads of bituminous macadam or cement concrete built within the last ten years on state highways within Worcester County have an average annual road cost of from \$3000 to \$3500 per mile The bituminous macadam type has a consistently lower annual



* Registration and License Fees plus 3 Cents Gasoline Tax
Federal Gasoline Tax and City Personal Property Tax not included

Figure 4. Annual Road Costs—Motor Vehicle Contributions—Traffic Route No 12—Leominster to Connecticut Line at Dudley, Massachusetts. BMA—Bituminous Macadam—Asphalt BCA—Bituminous Concrete—Asphalt. WB—Waterbound Macadam—Surface Treated BMT—Bituminous Macadam—Tar PCC—Plain Cement Concrete RCC—Reinforced Cement Concrete

road cost than cement concrete of the same width due mostly to the fact that the standard 8 in reinforced concrete pavement costs about half as much again as the standard 7 in bituminous macadam pavement

Annual traffic of approximately 1,000,000 vehicles is required in order that the contributions from registration, license fees and a 3 cent gas tax shall equal or exceed the annual cost of these 2-lane roads

(2) Modern, 3-lane pavements on state highways in Worcester County have an annual road cost of from $4000 to $5000 per mile, depending on the type, and require an annual traffic of approximately 1,500,000 vehicles in order that the contributions from registration, license fees, and 3-cent gas tax shall equal or exceed the annual road costs

(3) In general, the old types of pavement, i e , surface-treated water-bound macadam or gravel, have the lowest annual road cost, usually $2000 to $2500 per mile The first cost of these old pavements was low, therefore the annual interest and depreciation charges for these pavements are relatively low compared with their annual maintenance cost. This low annual cost is not a complete argument in favor of old types, however, because the vehicle operating cost is probably greater on these old types than on the modern types On Route No 12 the motor vehicle contributions greatly exceed the annual road costs for all of the old types, for Section 4 in Oxford the contributions are nearly double the road cost This particular section was reconstructed and widened during the latter part of 1933

(4) The character of the state highway system is constantly changing both as regards the type and condition of pavements and the traffic density The results presented for Route No 12 represent a "snap-shot" of that route in the year 1932 If the study were repeated in 1934, the results would be different because of the improvements which have been made

In the Sterling district the old pavements are now (1934) being replaced by wider and more modern surfaces coupled with improvements in alignment The new pavements will require less surface maintenance but there will be a considerable increase in interest on investment and depreciation When the exact costs are known it will be found that the annual road cost, represented by the full line on Figure 4, will be raised to well above the annual contributions in this Sterling section While the improvement may bring an increase in traffic, it will probably not be any considerable amount and therefore the contributions will not increase any considerable amount

In West Boylston the old BCA section is now (1934) being reconstructed and the 18-foot and 24-foot sections of BMA are being widened to 30 feet The latter sections were built in 1925 and 1928, respectively, and were assumed to have a life of 20 years After the widening has been made the road cost line in Figure 4 will be very much higher, probably approaching an annual cost of $5000 per year because this cost will include not only the interest and depreciation on the improvement but also on the original pavements, some of which had to be sacrificed to the widening The contributions line in this district will then be far below annual road cost line if the traffic remains at about 1,000,000 vehicles per year, an increase in traffic to 1,500,000 vehicles per year, however, would justify the improvement

The BMT section in Auburn is now (1934) being reconstructed Its annual maintenance has been very high The annual maintenance of the new pavement will doubtless be low, so that the increase in interest and depreciation on investment due to the betterment will be largely offset by the saving in annual maintenance costs, and the annual road cost line in Figure 4 will not be raised any considerable amount, it will still lie far below the contribution line

The BMT section in Oxford and Webster was reconstructed in 1933 The interest and depreciation on the cost of this betterment plus the probable annual maintenance of the new surface minus the rather high annual maintenance of the old surface will probably raise the annual road cost line up to a point close to the contribution line

It is obvious from this discussion that the present improvements in Sterling and West Boylston may not be wholly justified from the standpoint of contributions alone, but may be fully justified when one adds to it the favorable effect on vehicle operating cost, the old Sterling sections were the roughest of all those analyzed on Route 12 The BMA sections in West Boylston were widened to 30 feet in order to provide a pavement of uniform width throughout this district The recent improvement in Auburn is clearly justified because the annual cost will still lie below the annual contribution line The new surface in Oxford-Webster appears to be substantially justified

A diagram like Figure 4 can be used over a period of years by changing the lines upon it to conform to any considerable betterments or changes in annual maintenance cost or in annual traffic Whenever these changes are made the date can be recorded upon the new lines so that the diagram as a whole will indicate from time to time where improvements are clearly justified The phrase "clearly justified" is used because if contributions pay for the annual road cost the justification is apparent The improvement of the road surface, however, will decrease the vehicle operating cost, which is further evidence justifying the improvement, and in some instances it is clear that the decreased cost in vehicle operation due to the construction of a better surface may justify an improvement in which the line representing the annual road cost lies below the annual contribution line

The officials of the Highway Division of the Department of Public Works of Massachusetts have generously cooperated in providing basic data for this report. The figures compiled from these basic data,

however, have not been verified by the Department of Public Works nor have they had an opportunity to criticize them, therefore, the responsibility for their use lies wholly with the author

The assembly and analysis of the data and all computations and tables have been prepared by Mr Alexander J Bone who has also offered constructive suggestions which have in several instances been adopted without change

DISCUSSION

ON

COST ANALYSIS OF STATE ROADS IN WORCESTER COUNTY, MASSACHUSETTS

MR W A SHELTON, *U S Bureau of Public Roads* Who conducts the traffic counts in Massachusetts?

PROFESSOR BREED. This is done under Mr Taylor, Traffic Engineer of the Massachusetts Department of Public Works

MR SHELTON What length of watch do they use?

PROFESSOR BREED The usual length of count is 16 hours from 7 00 A M to 11 00 P M At certain key counting stations, however, 24-hour counts are made and these are used to expand counts at the 16-hour stations A complete traffic count is made of the State system every three years

DEAN ANSON MARSTON, *Iowa State College* From what source does the funds for the construction of these roads come Is it in part from a general property tax?

PROFESSOR BREED Roughly about one-third of it comes from registration fees and about two-thirds from gasoline taxes and a little from drivers' licenses and fines, etc Our total is a little over 20 million of which something like five million were diverted back to the cities and towns this year and roughly two million last year, and the rest of it spent in betterments, maintenance operations, policing, etc on a pay-as-you go basis, no bonds or property taxes

DEAN MARSTON In 1904 I was in touch with the Massachusetts State Highway Department and at that time automobiles were just starting to be numerous and the revenues must have been very small They had gone into quite extensive construction of roads, and I think they must have secured the main part of their money from other sources up to that time

PROFESSOR BREED: We have a state highway system of 1800 miles and our density of traffic on roads in Massachusetts, I believe, is the greatest in the United States. It is over 3000 vehicles per day per mile on the average. There are 61 per cent of our highways in the State system which carry upwards of 3000 vehicles per day. Such roads as the Newburyport turnpike running north from Boston toward New Hampshire carries an average of 8000 vehicles per day and about 25,000 on Sundays and holidays, and that 25,000 per day is quite comparable with the New Jersey viaduct. That road is a three and four lane road. Massachusetts has such dense traffic on so many miles that we get quite a lot of money.

DEAN MARSTON: You do not have to use general State sources?

PROFESSOR BREED: Not at all.

DEAN MARSTON. Has any study been made into this distribution of a certain percentage to State roads and a certain percentage to something else from the point of view of the source of money?

MR. BREED: I cannot answer that. Some of the State highway department officials have been quite interested in these studies we are making, and there is some hope that the State may make a State-wide analysis of highway costs. I feel personally that this is the type of study that could be made in cooperation with the Bureau of Public Roads because it would likely be of national value. I believe the problem would be simpler in Massachusetts than in most states because of the very complete record this State has of cost and of traffic.

THE ECONOMY OF HIGHWAY IMPROVEMENTS

BY HOWARD BURTON SHAW

Professor of Industrial Engineering, North Carolina State College

SYNOPSIS

In estimating the economy of contemplated road improvements, the question, "how much can we afford to invest now to save a determined amount of annual expense?" can be answered by comparing the additional investment with the saving in cost which it effects.

This method of computing economy is illustrated in detail by a project for improving a gravel road by surfacing it with concrete. For the cost data assumed it is shown that the improvement is justified for an annual traffic of 200,000 vehicles or more, but that for 100,000 vehicles it does not appear to be economical. The computation shows that the saving in annual road cost is relatively small in comparison with the saving in vehicle operating cost when the annual traffic is large. Indeed an approximate economy determination can be made by considering the vehicle cost only.

In the light of the old Roman saying that "the welfare of the people is the supreme law," the wonderful development of our highways shines as an outstanding example of public service for the benefit of the people. Viewed in that light, optimum service with economy should continue to be the guide

Economy determinations are valuable aids not only to business ventures, the main incentive for which is profit, but also for public activities such as the building of the nation's highways Economy for individual enterprises has not always lead to general prosperity or even to continued prosperity for the enterprises themselves

Likewise, economy studies are essential to highway development, but may not have predominating influence in the getting and expending of funds

It is apparent that stressing economy to the limit would defeat its own aims, for the roads carrying the most traffic would absorb the available funds and leave none for the subsidiary roads upon which much of the traffic must originate

The economy of an additional investment in a business enterprise can be determined from known data concerning amounts invested and yields before and after the investment and may be predicted with reasonable certainty, except for rapidly changing conditions

The yield is the annual operating revenue less the annual service cost, and the latter is the total of expenditures for operation, maintenance, insurance, and the annual provision for depreciation, which latter for highways is called the annual provision for "periodic maintenance "

The method generally used was stated by Arthur M Wellington nearly fifty years ago in his "Economic Theory of Railway Location " When the revenues are known the yields may be computed by subtracting the costs, and then the difference in yields may be compared with the difference in investments, or with the additional investment in an enterprise The percentage which the additional yield bears to the additional investment is a precise measure of the economy of the additional investment The economy may be determined by Wellington's method even when the revenues are unknown provided they are known to be equal, but when we attempt to use Wellington's method to arrive at the economy of highway improvements one of the first things that becomes apparent is that the operating revenue from highway transportation is not only unknown but by its very nature can never be known as an amount of money, because it is so largely in the form of service Also we know very little about the equality of the operating revenues before and after a highway improvement

The nearest measure we have of the operating revenue of a highway transportation system, consisting of public roads and private vehicles, is the annual traffic, the changes in which become intricate and uncertain in response to social and economic stimuli. Hence, we are forced to predicate the economy of highway improvement upon the annual traffic being the same after as before the improvement and base the computations on the reduction in cost, whereas increased revenue usually is the main incentive for investment in a business enterprise

With the revenue eliminated from consideration there are two ways in which we may proceed to compute the economy, which are designated as the "Total Annual Cost Method," and the "Saving in Annual Expense Method"

The Total Annual Cost Method The interest upon the total investment is computed and added to the annual service cost to arrive at total annual cost, and then total annual costs may be compared. This is the method used by the Committee on Highway Transportation Economics in its 1929 and 1930 reports It took much time and effort to determine the "cost to construct" for short sections of roads, so the suggestion has been made that an approximation of the amount invested by a State in its highway system could be arrived at in other ways

Saving in Annual Expense Method We may compare the investment to be added in making an improvement with the difference in annual service costs before and after the improvement, that is, compare the additional investment with the saving in cost which it effects This second method is the one used in this report in an attempt to answer the question "how much can we afford to invest now to save a determined amount of annual expense?" An answer to this question may be stated as follows. "The economy will be shown by computing the percentage of saving to the cost of the improvement," because the saving is the only return upon the investment which we can compute.

To illustrate this method of computing economy, consider an improvement by surfacing a gravel road with concrete Data are assumed approximately as given by Mr A C Benkleman for the State of Michigan, in his paper, "Demonstrating the Economy of Good Roads," in Civil Engineering for July, 1933

The cost of surfacing a mile of gravel road with concrete is estimated to be $15,000 The average annual maintenance for the gravel road is estimated to cover periodic maintenance The annual cost for "periodic maintenance" of the concrete road is computed as the $4\frac{1}{2}$ per cent annuity for replacement of the concrete surface every 25 years at a cost of $15,000 per mile

The cost of operating an automobile for a mile is estimated to be 8 mills less on a concrete surface than on a gravel road This estimate of saving per vehicle mile is taken from Figure 1, page 87 of the 1932 Proceedings of the Highway Research Board, "A Study of Costs on Various Types of Highway," by Raymond G. Paustian.

Table I sets out these data and the computations:

From Table I it is easy to see that the improvement is justified for an annual traffic of 200,000 automobiles, and in increasing measure up to 750,000, for which the annual saving is over 40 per cent of the cost of the improvement For an annual traffic of 100,000 automobiles the improvement appears to be non-economic, and an annual traffic of 150,000 automobiles is on the border line and does not so amply justify the improvement as a larger annual traffic It is assumed that the percentage of saving to investment should be at least twice the current rate of interest, say 10 per cent, in order to promise real economy

TABLE I

COMPUTATION OF ECONOMY FOR CHANGE FROM GRAVEL ROAD TO CONCRETE SURFACE PER MILE OF ROAD

(The annual traffic is estimated to be the same after as before the improvement)

Number of automobiles per year	Annual Road Costs				Annual Saving			
	Gravel Road	Surfaced with Concrete						
	Annual Service cost, i e, average annual cost of maintenance	Average annual maintenance	Annual cost for periodic maintenance 4½% annuity for replacement every 25 years at cost of $15,000	Annual Service Cost	Road Cost	Vehicle Cost	Total	Percentage of $15,000
100,000	$880	$328	$336	$664	$216	$800	$1,016	6 77
150,000	954	347	336	683	271	1,200	1,471	9 8
200,000	1,020	360	336	696	324	1,600	1,924	12 8
400,000	1,130	440	336	776	384	3,200	3,554	23 7
750,000	1,485	575	336	911	574	6,000	6,574	43 8

In order to have accuracy in computations by this method it is necessary to have accurate data as follows

(a) The average annual maintenance cost for each type of surface This may be compiled from records of costs of similar maintenance

(b) A reasonably good estimate of the life of each type of surface and the cost of replacing it

(c) The present annual traffic, rather closely estimated and classified as to type of vehicle

(d) An estimate of the difference in vehicle-mile cost on the two types of road surface, determined as accurately as possible

More dependence is to be placed on the difference than on the vehicle-mile costs themselves The difference should be accurate to less than a mill, because a variation of one mill with an annual traffic of 750,000 automobiles means a variation in annual saving of $750 per mile, which overshadows the saving in road costs

The computations also show that the saving in annual road cost is relatively small in comparison with the saving in vehicle cost, when the annual traffic is large, for an annual traffic of 750,000 automobiles the saving in vehicle cost is computed to be $6,000 out of a total saving of $6,574 Consequently, the economy of changing to a higher type of road surface is mainly dependent upon the amount of traffic and the difference in vehicle costs on the different surfaces

In fact, the saving in road cost on changing to a higher type of pavement is relatively so small that we may get an approximate economy determination by considering the saving in vehicle cost only Thus, if the vehicle cost per mile is 8 mills less on the next higher type of pavement, an annual traffic of 400,000 vehicles per year will mean a saving in annual vehicle cost of $3,200 per mile of road due to the change, and this capitalized at 10 per cent gives a permissible expenditure of $32,000 per mile, whereas the change can probably be made for $15,000 per mile

The larger the annual traffic, the greater is the justification for changing to the next higher type of road surface irrespective of the investment, or the "cost to construct" the existing road, and to a considerable extent irrespective of the annual road cost

This balancing of the saving in annual expense against the cost of making the improvement appears to be applicable particularly to the computation of economy for individual projects for the improvement of existing highways without taking into account the additional service which may result from an increase in annual traffic on a section of road which is improved or on the highway system of the State Such increased service might well prove to be more compelling than the saving in vehicle cost only

No attempt is made to apply this second method to the study of state highway systems, for which the first method appears to be applicable with some modifications

A modification of the committee's formula for annual road cost suggests itself, and that is to compute the interest upon the investment rather than on the "cost to construct," because the investment probably can be found from the records of expenditures, and then make deductions for assets retired or no longer useful, very much in the same way that "fixed investment" is kept for a public utility.

REPORT OF PROJECT COMMITTEE ON TRACTIVE RESISTANCE AND ALLIED PROBLEMS

W E Lay, *Chairman*

Professor of Mechanical Engineering, University of Michigan

FURTHER TRACTIVE RESISTANCE TESTS WITH A GAS ELECTRIC DRIVE AUTOMOBILE

By Raymond G Paustian

Junior Highway Engineer, Iowa Engineering Experiment Station

[In Abstract[1]]

Further tests on the gas-electric drive automobile described at the Twelfth Annual Meeting of the Highway Research Board are given in this report The equipment and methods used are described in the Twelfth Proceedings of the Highway Research Board [2] For these tests the Model 314 Cadillac Coach with General Electric gas-electric drive was used with an average weight, including driver and two observers, of 6300 pounds The tires were 33 by 6 75 inch heavy duty balloons, normal inflation 45 lb per sq in , load per tire 1575 pounds.

Several methods[3] of measuring tractive resistance have been described To them should be added the method by direct measurement used in this investigation In this method the magnitudes of the forces resisting movement of the vehicle are determined by measuring the electrical energy consumed in propelling the car

A series of runs was first made on a level concrete road surface at a uniform temperature of 70°F to determine the amount of power required to drive the test car at various speeds From these tests it was found that the total tractive resistance (rolling + air resistance) could be expressed by the formula

$$R = 45.0 + 1\ 6125S + 0\ 025875S^2$$

where R = Total tractive resistance
S = Speed in miles per hour

The correctness of this formula was verified by a series of determinations of the tractive resistance of the test car by means of the "Coasting Method"[3] using one, two and three per cent grades The

[1] A detailed report of this investigation may be found in a forthcoming Bulletin of the Engineering Experiment Station, Iowa State College

[2] Tractive Resistance Determinations with a Gas Electric Drive Automobile, R G Paustian, Proc Highway Research Board, Vol 12, page 75

[3] Air Resistance of Motor Vehicles, W E Lay, Proc Highway Research Board, Vol 12, pp 66–75

results of these tests were corrected for the effects of weight variation, temperature and wind in so far as is possible at present The values for various speeds are shown in Figure 1

Having thus established the accuracy of the determinations the total resistance was then separated into its component parts, rolling and air resistance, by chassis dynamometer tests of rolling resistance Efficiency curves plotted from the results of the dynamometer tests indicated what percentage of total power at the drive shaft is used in overcoming rolling resistance and what amount is available for use in

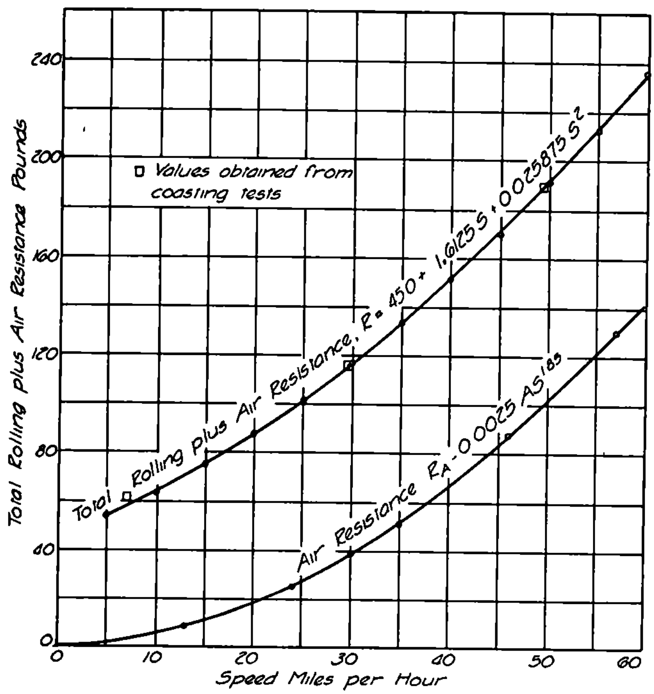


Figure 1 Analysis of tractive resistance on a level road surface. Smooth concrete surface. Temperature 70°F.

overcoming air resistance on the road The following formulas applicable to the particular car in use were deduced from these tests:

$$\text{Rolling Resistance} = R_r = 45 + 3\ 189S^{0\ 676}$$
$$\text{Air Resistance} = R_a = 0\ 0025AS^{1\ 85}\ (\text{Fig } 1)$$

where A = projected area of car (28 72 sq ft in this case)

Having these fundamental characteristics established, investigations were made of power and gasoline consumption on grades, energy consumption on rolling grades, temperature effects, wind effects, and tire behavior.

POWER AND GASOLINE CONSUMPTION ON GRADES

On ascending grades, to the rolling and air resistance must be added the grade resistance due to the component of the weight of the vehicle down the grade. It is equal to 20 lbs per ton of vehicle weight times

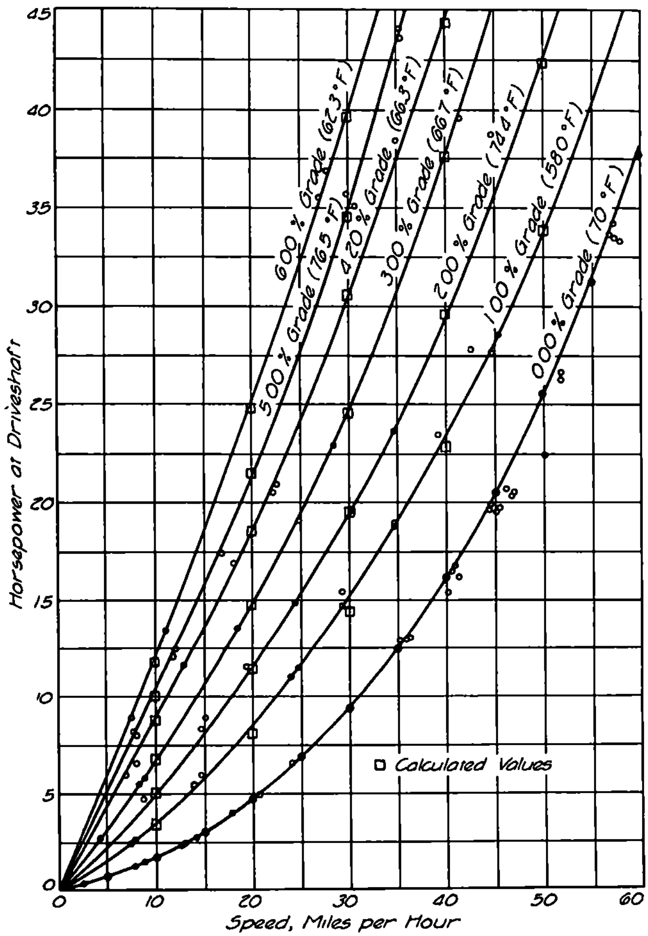


Figure 2. Power requirements on grades Concrete pavement

the per cent of grade The amount of power required to overcome the grade resistance can be calculated from the formula

$$\text{H P} = \frac{R\ S}{375}$$

where R = total resistance and S = speed in miles per hour The results of tests on individual grades from one to six per cent (Fig 2) show exceedingly close agreement between the measured and calculated values of rolling plus air plus grade resistance An interesting feature of these tests is the indication that there is a definite relation between rate of grade and the speed at which the greatest mileage per gallon of gasoline is secured (Fig 3)

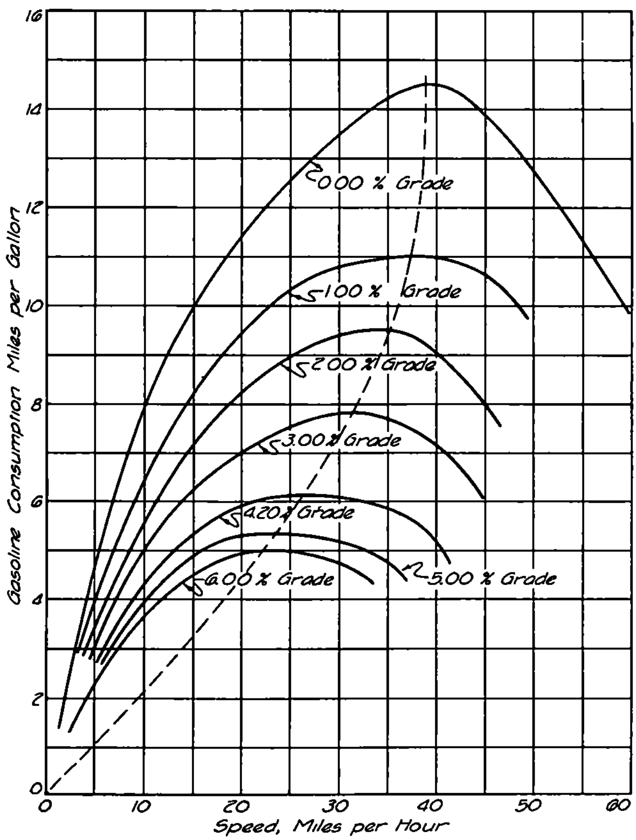


Figure 3. Gasoline consumption on grades Concrete pavement

Energy Consumption on Rolling Grades

In order to secure data related to ordinary driving conditions, tests were made over roads having series of ascending and descending grades, thus bringing in the effects due to momentum The record was secured by taking photographs of the instrument panel

Most of these runs were made at constant throttle opening although data were also secured for constant speed operation and by allowing the car to coast down and then ascend at a constant speed (called minimum speed) The operation with constant throttle openings showed a definite increase in speed over that which the same throttle openings gave on the level surface Constant throttle opening was accompanied by constant power consumption irrespective of the road profile

A surprising result of the constant throttle opening was that the average amount of power used in traveling over a rolling grade at a certain average speed was less than that required to propel the car over a level surface at the same speed (Fig 4)

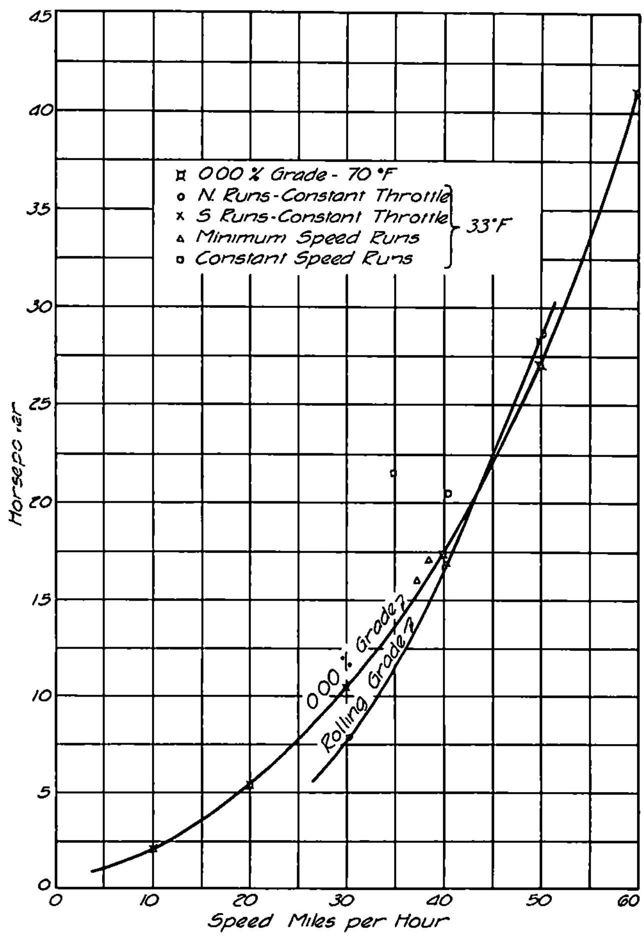


Figure 4. Comparison of power requirements on rolling and level grades

The "constant speed" and "minimum speed" runs showed increases in power consumption over those used on the zero per cent grade The increase in the average power used at constant speed may be accounted for by the fact that the brakes are used while descending and excessive power was needed in ascending The average amount of power used in the minimum speed runs was less than that used at constant speed and more than that needed for constant throttle

TEMPERATURE EFFECTS

The results of the tests indicate definitely that power consumption increases with decrease in temperature and that the effect increases with speed At 45 miles per hour the increase in power caused by a temperature drop of from 57°F to 21°F was 3 75 H P

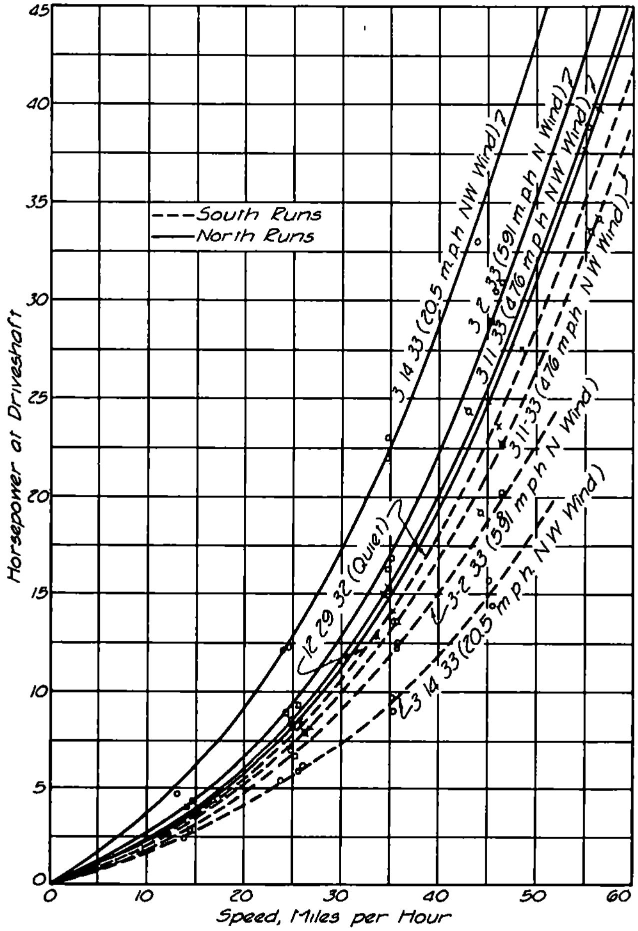


Figure 5 Effects of wind on power consumption

EFFECTS OF WIND

A number of interesting runs were made showing the effects of head winds and tail winds of various velocities upon power consumption Wind velocities were measured by a pitot-static tube mounted on the front of the car and connected with a manometer tube on the instrument panel Typical relations are shown on Figure 5 The differences between the curves for runs in opposite directions was due to the fact that the course used was not exactly level

BEHAVIOR OF TIRES

Observations of the relations between speed, grades, tire size and tire temperatures under test conditions yielded some interesting facts

The diameters of the tires increased with speed, both on level and ascending grades

At a given speed the diameters of the rear tires were less than those of the front tires On a level road the rear tire diameters increased 0 35 in and front tire diameters increased 0 41 in while accelerating from 0 to 50 miles per hour

Tire temperatures were measured with a special thermometer set in a protective case within the inner tube

The temperature within the tire rises rapidly after the car has run a short distance and continues to rise until a maximum for the speed is reached Rear tire temperatures are considerably greater than those of front tires A close relation between tire and air temperatures was also noted

For the particular tires used in these tests the following relations prevailed·

Front Tire Temperature = Air temperature (Deg F) + 8 + 0 22 times speed (m p h)

Rear Tire Temperature = Air temperature (Deg F) + 18 + 0 33 times speed (m p h)

It was also noted that the tire temperatures were sensitive to the difference between sunlight and shade

DISCUSSION
ON
TESTS WITH A GAS-ELECTRIC DRIVE AUTOMOBILE

MR F LAVIS, *Consulting Engineer, New York* The observations of these tests confirm those made and reported by the late A M Wellington in his "Economic Theory of Railway Location" a good many years ago to the effect that, within certain limits, rolling grades had practically no effect on the costs of train operation

It is evident that rolling grades on highways which do not affect costs of operation may be much more pronounced both as to length and steepness of gradients than those of railways The limits on railways are, of course, those descending gradients which require the use of brakes, or where the additional effort required on ascending gradients is not balanced or nearly so by power saved on descending gradients

In the studies[1] made in connection with the design of Route 1 Extension, now Route 25, of the New Jersey State Highways, the assump-

[1] Highways as Elements of Transportation, Transactions Am Soc C E Vol 95, p 1020 (1930)

tion was made that within certain limits rolling gradients had no effect on costs of operation of the vehicles using the highway We had then no facts or experiments on which to rely so we were obliged to make an arbitrary ruling Mr Paustian's tests now carry our knowledge a little further

Mr T C Smith I noticed that wind resistance was made a function of cross-sectional area, but that there was no factor with regard to stream-lining What does that area mean?

Mr Paustian The area referred to is the projected or cross-sectional area of the car The effect of stream-lining will show up in the value of the constant, K, used in the equation for air resistance For our test car, this value is 0 0025, for a car that is more stream-lined, there will be a corresponding lower value of the constant

SKIDDING CHARACTERISTICS OF ROAD SURFACES

By R A Moyer[1]

Associate Professor of Highway Engineering, Iowa State College

SYNOPSIS

The coefficients of friction of rubber tires on various road surfaces both wet and dry were measured for both straight ahead and sideways skidding at speeds of from three to forty miles per hour An ingenious special integrating dynamometer was designed for measuring the skidding forces

Tests were run upon 25 different types of surfaces, including asphalt, tar, road oil, portland cement, brick, gravel, cinders, asphalt plank, steel plates, wood plank, and mud on concrete In general a marked decrease in coefficient of friction was noted with increase in speed although the reverse was true in the case of the gravel and cinder surfaces It was found that the coefficients at three to five mile speeds are not indicative of the values at the higher rates of speed

Typical of the data observed in these tests are the following coefficients of friction on various wet pavement surfaces for skidding straight ahead at thirty miles per hour sandstone rock asphalt 0 59 to 0.77, sheet asphalt 0 47 to 0 63, bitulithic 0 50 to 0 63, asphaltic concrete 0 65 to 0 60, asphaltic retread 0 40 to 0 51, road oil mix 0 35 to 0 50, penetration macadam 0 20 to 0 28, repressed brick with grout

[1] The project covered by this report is a continuation of the program of highway research initiated 14 years ago by Dean T R Agg at Iowa State College By tests made in 1923 and 1927 fundamental facts concerning the coefficients of friction of tires on road surfaces were established However in view of the changes in tires, road surfaces and traffic conditions further studies were begun by Professor Moyer two years ago Mr Earl Allgaier and Mr Donald Berry, to whom much of the credit for the success of this project should be given, assisted the author throughout most of the work

filler 0 35 to 0 48, vertical fiber brick with asphalt filler 0 38 to 0 52; portland cement concrete, rough finish 0 37 to 0 48; portland cement concrete, smooth finish 0 40 to 0 46, medium hard tar macadam 0 66 to 0 47, oiled gravel 0 45 to 0 46, untreated gravel 0 68 to 0 71. (The first figure in each case is for smooth tread tires, the second for non-skid treads) A distinguishing characteristic of the bituminous pavements that showed the greatest resistance to skidding was the "sand paper" texture of the surface From a theoretical analysis it is deduced that to be reasonably free from the danger of skidding a road surface when wet should have a straight skid coefficient of 0 4 or higher at forty miles per hour and a static or side skid coefficient of 0 5 or higher at thirty miles per hour Although an attempt was made to select representative surfaces in typical condition, the results reported in this paper apply only to the ones tested and it should not be assumed that the results of tests on a particular surface are typical of all surfaces in that class

Many tests are also reported showing the effects of tire pressure, wheel loads, type of tire tread and temperature

There are three distinct forms of skidding on roadway surfaces, (1) straight skidding in the direction of travel with the wheels locked, (2) impending skidding, which is a modified form of straight skidding in the direction of travel with the wheels just at the point of complete sliding, and (3) skidding sideways in a direction normal to the line of travel. The first form is encountered when brakes are applied suddenly causing the wheels to lock and slide The second form is obtained by applying the brakes gradually to the point where the wheels are still turning but where skidding is imminent The third form is encountered on curves that are not superelevated enough for the speed at which the vehicle is traveling, or when passing other cars on tangents

The relationship between the three forms is best expressed in terms of the coefficient of friction, which is, the ratio of the force causing the tires to skid to the load on the tires If a large force is necessary to cause the tires to skid the coefficient will be large, but if the force is small the coefficient will likewise be small The straight skid coefficient of friction, generally referred to as the kinetic coefficient of friction by writers on mechanics, is obtained when sliding takes place in the line of travel The coefficient referred to as the static coefficient by writers on mechanics, is that obtained by applying the brakes gradually to the point where skidding is impending in the line of travel The side skid coefficient of friction is obtained when the tires are skidded sideways

TEST METHODS AND EQUIPMENT

The measurements of the coefficients of friction were made on road surfaces as nearly as possible under the conditions encountered by traffic Since the power requirements for towing a full sized car would greatly restrict the field for testing, a two-wheel trailer test unit was so constructed that it could be used interchangeably for the three

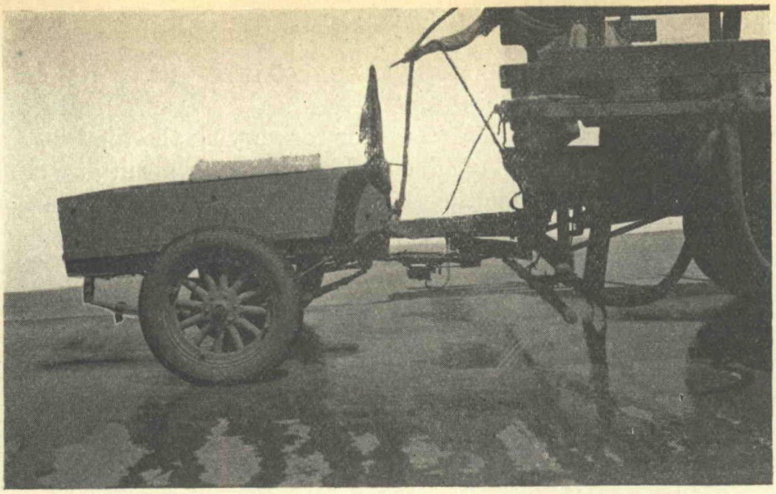


Figure 1. Arrangement of Equipment for Straight Skid Tests

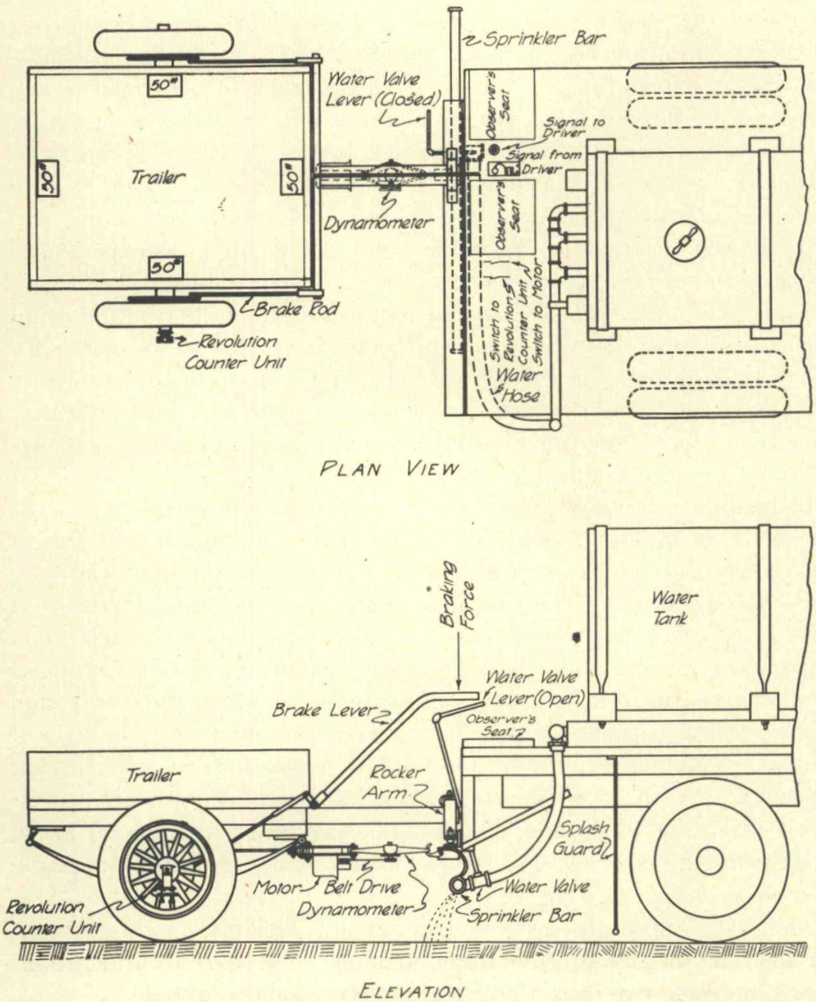


Figure 2. Arrangement of Test Equipment Used for Determining the Coefficient of Friction of Tires on Road Surfaces When Skidding Straight Ahead

forms of skidding Provision was made for varying the total load on the trailer from 630 to 1630 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 ranging from 3 to 40 miles per hour

Since wet surfaces provide the most common dangerous skidding condition, a water tank and sprinkling system were mounted on the high-speed truck which towed the skidding trailer The surface was sprinkled directly in front of the test trailer 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 2) 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 Bendix 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, at first when running tests for impending skidding, it was found that uniform braking distribution could not be maintained between the two wheels, partly due to the inequality in the braking force on each wheel and partly due to the differences in 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 3) such that the longitudinal axis of the trailer made an angle of about 15 degrees with the line of motion of the towing truck The 15-degree angle was selected on the basis of 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 maximum 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 wheels rolled forward with a side skid motion, the trailer tending to swing into the direction of travel of the tow truck, thus simulating the action of a car skidding on a curve

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 as completely as possible, (3) it should measure accurately to within 10 pounds, forces ranging from 100 to 2,000 pounds, (4) the forces should be meas-

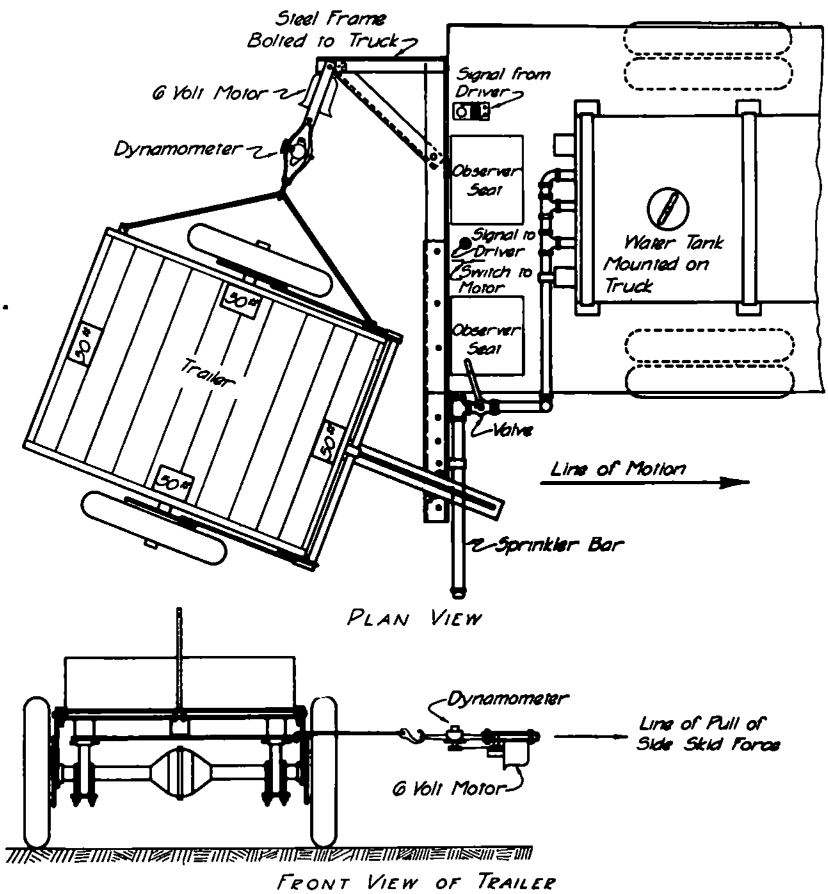


Figure 3. Arrangement of test equipment used for determining the coefficient of friction of tires on road surfaces when skidding sideways.

ured directly to eliminate the possibility of lag or the setting up of inertia 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) the making of frequent field checks of the calibration was desirable

The integrating device designed for use with the integrating dynamometer consisted of a rotating fibre disc and two revolution counters The fibre disc was mounted at a fixed distance from one side of a

Kohlbusch dynamometer spring. One of the revolution counters was geared to the disc and recorded its revolutions to the nearest tenth of a revolution. The disc was rotated at a constant speed by a motor drive (Fig. 4). 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 fibre disc. When pull was applied to the dynamometer, the steel wheel moved toward the center of the disc, and turned with fewer revolutions in proportion to the number of revolutions made by the disc. The quotient obtained by dividing the number of revolu-

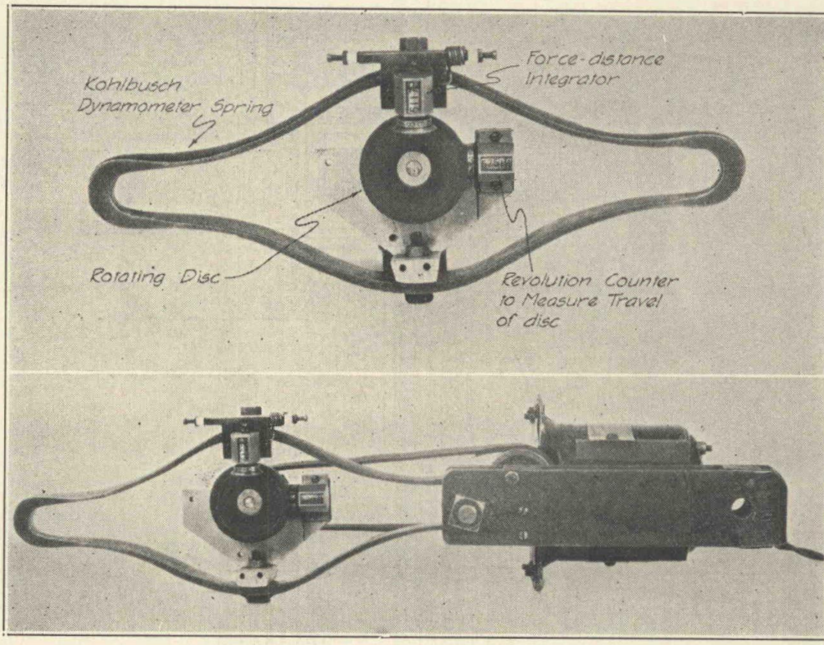


Figure 4. Integrating Dynamometer and Dynamometer with Motor Drive

tions of the disc by the number of revolutions made by the steel wheel was a measure of the pull transmitted by the dynamometer.

A typical calibration curve is shown in Fig. 5. 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. At first, some difficulty was experienced in keeping the steel wheel from slipping on the disc. This source of error was eliminated by slightly roughening the edges of the steel wheel. The field calibration for zero load served as a valuable 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 indicated the ruggedness and dependability of the dynamometer.

All of the tests were run at uniform speed, on a smooth level stretch of road, preferably, when traffic permitted, in the center of the road to obviate the effects of crown, and also where no change in the surface condition could be observed From two to four runs were made in each direction at speeds of 3, 5, 10, 20, 30, and 40 miles an hour. In

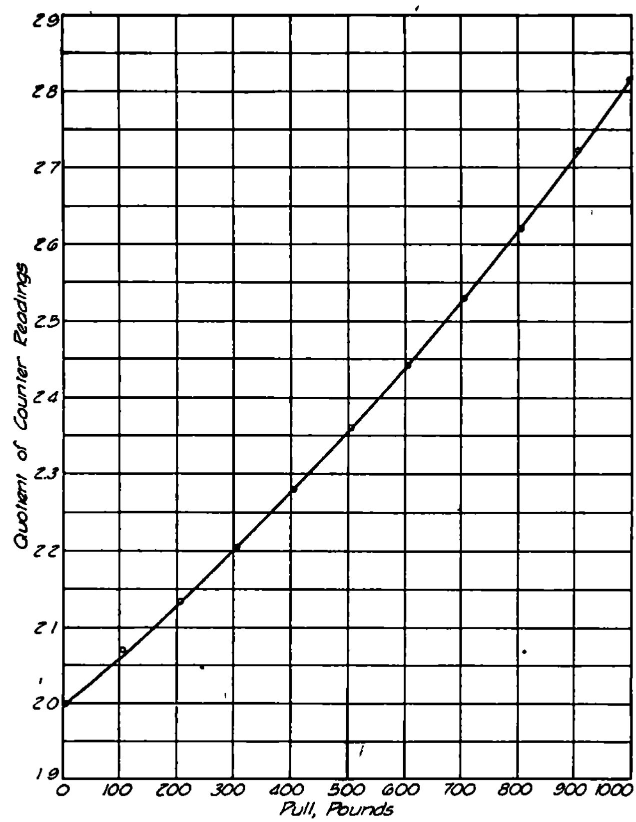


Figure 5. Calibration Curve for Integrating Dynamometer. October 28, 1933

a typical test run on a wet surface, the course was given a preliminary sprinkling and the trailer was then brought into position to start the run The two counters were read and the signal to start the run given A signal from the driver of the tow truck indicated when the desired test speed was reached The observer then started the dynamometer motor, and after skidding the tires over 50 to 150 feet of road surface, depending on the speed at which the test was run, shut off the motor and signaled the driver to stop The readings on the counters were again taken. The quotient of the differences in the two readings was the measure of the average pull transmitted by the dynamometer for the duration of the test.

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 previous tests 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.

All of the tests in the comparison of road surfaces were made with popular brands of new tires, 19 by 4.75 inches, of the open tread design

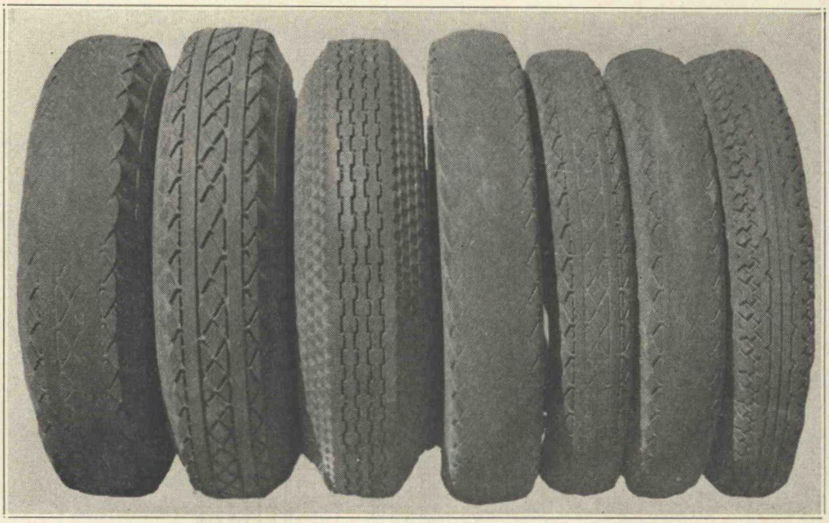


Figure 6. Tires used in Skid Tests

and similar tires with treads worn smooth. A tire pressure of 35 pounds per square inch was maintained during the tests.

To determine the effect of tire size and tread design on the coefficient of friction, tests were run with three sizes of tires, 16 by 7.00 inches, open type treads, 15 by 7.50 inches, closed type tread, and 19 by 4.75 inches, closed type tread (Fig. 6). This series of tests was also made with both new and smooth treads.

The relation between the percentage of slippage of the trailer wheels and the coefficient of friction when the braking force on the trailer was increased by uniform increments to the point at which continued sliding in the direction of travel could be maintained was determined by special apparatus.

Revolution counters which could be read to one-tenth of a wheel revolution, were mounted on the hubcap of the right wheel of the trailer and the right front wheel of the tow car The counter units were suspended in such a manner that they remained stationary as the wheels revolved The counter on each unit was fastened to one end of the lever arm and held in contact with the hub of the wheel by a coil spring An electromagnet was used to disengage the counter The electromagnets of the two units were wired in series so that the counters could be started and stopped at the same time

An ordinary tire gauge, calibrated with a standard pressure gauge, was used for measuring tire pressure

Pavement temperatures were measured with a specially constructed mercury thermometer, fitted into a small rubber cup and placed in direct contact with the road surface Tire temperatures were measured by inserting the bulb of a mercury thermometer into the tire tread, shielding it from air currents with a heavy felt pad.

In the early stages of the test work, a Chrysler sedan was used as a tow car However, to meet the requirements for sprinkling the surfaces and to provide the power necessary to run tests at high speeds, a high speed $2\frac{1}{2}$ ton Graham truck, was used

CALCULATION OF THE COEFFICIENTS OF FRICTION

The coefficients of friction can readily be computed from formulas based on consideration of the forces acting on the trailer In the straight skid tests the forces acting on the trailer are shown in Fig 7 The weights were placed in fixed positions so that the trailer was in balance during normal operation on a level surface The frictional force of the tires on the road surface caused a transfer of part of the weight of the trailer from the wheels to the point where the tongue of the trailer was fastened to the tow car This weight transfer is represented by the force T in Fig 7 By taking moments about the point of contact of the tires and the road surface, the relation between T and the pull (P) transmitted by the dynamometer is obtained

$$T = \frac{17P}{60}$$

The coefficient of friction for straight skidding, and also for impending skidding, can then be computed by means of the following formula based on the definition of the coefficient of friction:

$$f = \frac{P}{W - 0\,283P}$$

Since the pull of the dynamometer in the side skid tests was applied at a point about 24 inches above the road surface, a weight transfer from one wheel of the trailer to the other resulted This weight transfer

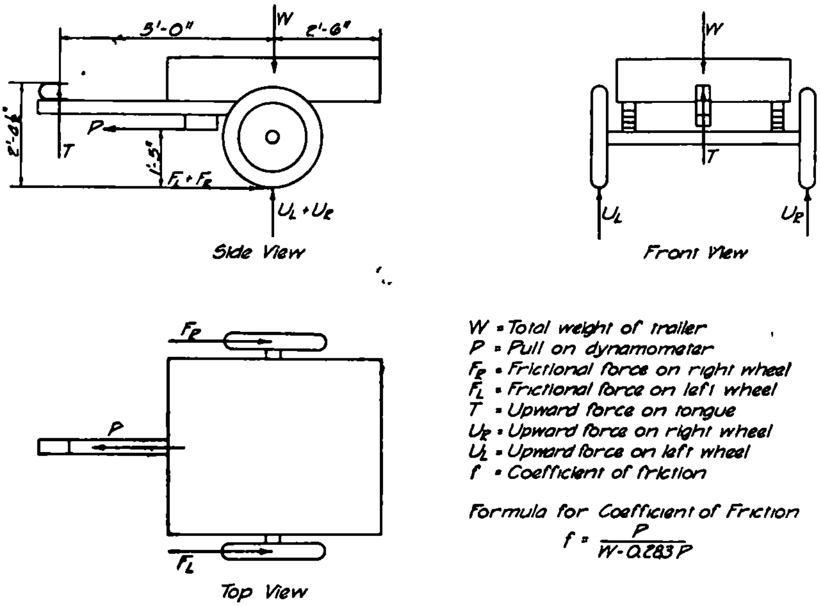


Figure 7. Diagram of Forces Acting on Two-wheel Trailer When Skidding Straight Ahead.

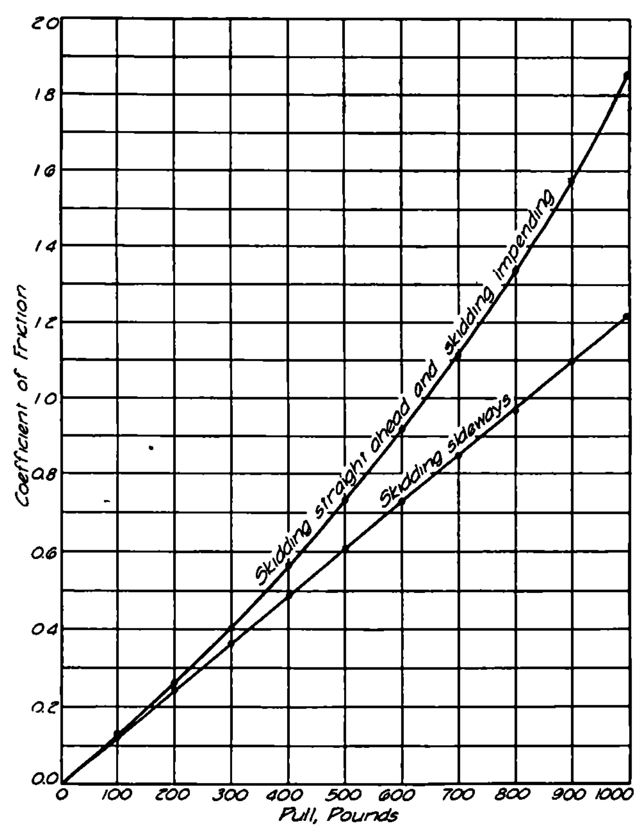


Figure 8 Relation of Pull Measured by Integrating Dynamometer to Static, Side and Straight Skid Coefficients for Calibration of October 28, 1933.

was very similar to that for a car, with a low center of gravity, skidding on a curve not sufficiently superelevated However, since the entire weight of the trailer was carried by the two wheels, the coefficient of friction for skidding sideways is obtained by dividing the pull measured by the dynamometer by the total weight of the trailer:

$$f = \frac{P}{W}$$

The relationship between the coefficient of friction and the pull measured by the dynamometer in the various types of skid tests is shown graphically in Fig 8.

RESULTS OF TESTS

Probably the most important phase of this investigation was the determination of the skidding characteristics of 25 representative road surfaces Since wet surfaces are the most hazardous type ordinarily encountered, a majority of the tests were run on wet surfaces For comparison, however, tests were also run on eight representative dry surfaces

Four series of tests were run on each surface, straight skidding with new tread and smooth tread tires, and skidding sideways with new tread and smooth tread tires These tests provided data for a critical analysis of the skidding characteristics of the surfaces and also, since the relative standing of the surfaces in all four tests was not altered materially served as a valuable check on the test methods The tests were run during June, July, and August, when the changes in air temperature at the time of testing were not great enough to require corrections in the coefficients

The coefficients of friction for the 25 surfaces when wet are shown in Figures 9a, 9b, 10a, 10b, 11a, 11b, 12a, 12b. The key for the surfaces in these figures is given in Table 1. It should be observed that the letter in the key indicates the type of surface, that is, A for asphaltic and road oil surfaces, B for brick, C for concrete, etc.

An examination of the curves will disclose a wide range of values for the coefficients of friction for the various surfaces and a marked *decrease* in coefficient as the vehicle speed increases. Only in the case of gravel and cinder surfaces is a definite *increase* in the coefficient obtained with an increase in speed. In these cases the wheels plow into the gravel or cinders as the skid progresses, thus providing mechanical resistance in addition to frictional resistance. The change in coefficient of friction due to changes in speed was more marked in the case of some surfaces than in others It is now evident that the coefficients for low speeds are not a true indication of those for the higher speeds This is best illustrated in the case of the penetration macadam surface which has a relatively high coefficient at 3 miles per hour and a dangerously low coefficient at 40 miles per hour In practically all the curves there is a definite flattening out with an increase in speed, and the trend for all of the surfaces seems to be fairly well established

A surprising feature of the tests on wet surfaces was the results obtained in the tests on black top surfaces The coefficients obtained on the high type asphaltic surfaces were consistently higher than the coefficients on any of the other types tested This was especially true for the sheet asphalt, rock asphalt, and asphaltic concrete surfaces While running the tests, it was apparent that the "sand-paper" finish on these surfaces was partly responsible for their high resistance to skidding There was also a definite indication that the hardness of the asphalt and tar binders was responsible for the higher coefficients for the as-

TABLE I

KEY FOR SURFACES IN FIGURES 9, 10, 11, 12 AND 13

A-1	Asphaltic Concrete, Ames, Iowa
A-2	Rock Asphalt, Sandstone type, U S 31, S of Kokomo, Indiana
A-3	Warrenite Bitulithic, Ames, Iowa
A-4	Sheet Asphalt, Des Moines, Iowa
A-5	Asphalt Retread, Ind 39, S of Frankfort, Indiana
A-6	Road Oil Mix, Ind 26, East of Kokomo, Indiana
A-7	Oiled Gravel, Iowa 60, W of Ames, Iowa
A-8	Bituminous Mulch Top, Ind 26, E of Lafayette, Indiana
A-9	Penetration Macadam—with soft seal coat Ind 29, S of Logansport, Ind
B-10	Repressed Brick with grout filler, Story City, Iowa
B-11	Vertical Fiber Brick with *no* excess asphalt filler, Des Moines, Iowa
B-12	Vertical Fiber Brick with excess asphalt filler, Des Moines, Iowa
C-13	Portland Cement Concrete, rough finish, U S 30, W of Ames, Iowa
C-14	Portland Cement Concrete, smooth finish, U S 65, N of Des Moines, Iowa
C-15	Portland Cement Concrete, rough finish, Ind 22, E of Kokomo, Indiana
G-16	Untreated gravel, County Road, S of Ames, Iowa
G-17	Cinders, Ames, Iowa
A P -18	Mineral Surfaced Asphalt Plank, U S 34, W of Fairfield, Iowa
A P -19	Fine Aggregate Type Asphalt Plank, U S 6, W of Atlantic, Iowa
W P -20	Maple Hearts Wood Plank, Iowa 3, E of Clarinda, Iowa
S P -21	Steel Traffic Plates, 13th St Bridge, Ames, Iowa
M C -22	Mud on Concrete, Ames, Iowa
T-23	Ohio Tar Macadam, Ohio 11, E of Eaton, Ohio
T-24	Ohio Hard Tar Surface Treatment, Ohio 4, N of Middletown, Ohio
T-25	Ohio Medium Tar Surface Treatment, Ohio 123, S of Franklin, Ohio

phaltic, road oil, and tar surfaces In the tar surfaces, the highest coefficients were obtained on the high type tar macadam surface slightly lower coefficients were found for the hard tar surface treatment. and the lowest coefficients occurred with the medium tar surface treatments It is interesting to note that the rough-textured "non-skid" finish on the medium and hard tar surfaces did not provide as high a coefficient as the comparatively fine textured finish of the old tar macadam The rough-textured "non-skid" finish in the asphalt retread surface did not raise the coefficient materially over the road oil mix type which had a finer surface texture The coefficients of all of the so-called "non-skid" surfaces were lower than the coefficients for the rock asphalt surface which had a fine "sand-paper" finish It would appear reasonable to conclude then that the rough, coarse-textured "non-skid" finish, frequently placed on asphalt and tar surfaces, is not necessary for high resistance to skidding A "sand-paper" finish with a relatively hard asphalt or tar binder should serve equally well and should reduce the excessive tire wear generally attributed to the "non-skid" type of finish.

However on certain types of low cost bituminous roads where a tendency has been observed for the bitumen to flush to the surface under the action of traffic, the use of a modified form of the coarse texture type of non-skid surface, may be advisable The use of soft asphalts, soft tars or light road oils and the lack of control of the amount of bitumen usual in these types makes the transition from the "sand-paper" finish to a smooth slippery surface relatively easy By using a modified form of non-skid finish in which the size of cover stone is limited to one-half inch maximum, this transition should be made more difficult without undue increase in tire wear

Although the highest coefficients for wet surfaces were obtained on high type asphaltic surfaces, the coefficients obtained on the asphalt penetration macadam with soft seal coat and the fine aggregate type asphalt plank were among the lowest coefficients measured A maintenance patrolman reported frequent skidding accidents on the penetration macadam when wet This report checks with the findings of these tests It is interesting to observe that by providing a mineral surfacing on asphalt plank, the coefficients of friction were raised to the values obtained for wet portland cement concrete surfaces

The three types of wet portland cement concrete surfaces had the most consistent coefficients among the surfaces tested It should be noted, however, that the coefficients for wet concrete were 15 to 40 per cent lower than for the wet high type tar and asphalt surfaces tested The concrete surfaces in Iowa, also in Indiana, with the rough finish were still relatively new The surface with the smooth finish had been in service for about 12 years The coefficients for similar test conditions on these surfaces varied only within very narrow limits, and the results for the three surfaces are represented by the same curve in Figures 9a, 10a, 11a, and 12a

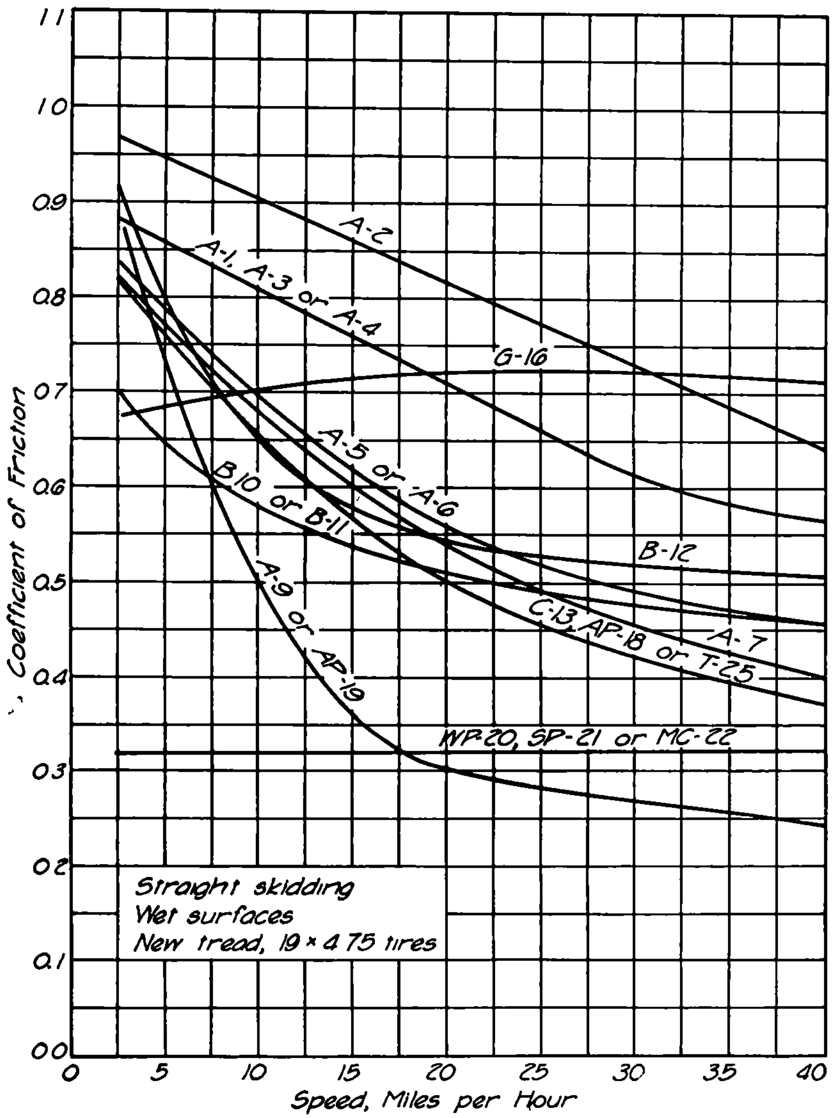


Figure 9a. Coefficients of Friction of Tires Skidding Sideways on Wet Surfaces. New Tread Tires. See Table I For Key to Surfaces.

The coefficients for the brick surfaces when tested wet were slightly lower than those obtained for the wet portland cement concrete surfaces The coefficients for the three wet brick pavements were from 10 to 20 per cent lower in the straight skid tests and 40 to 50 per cent

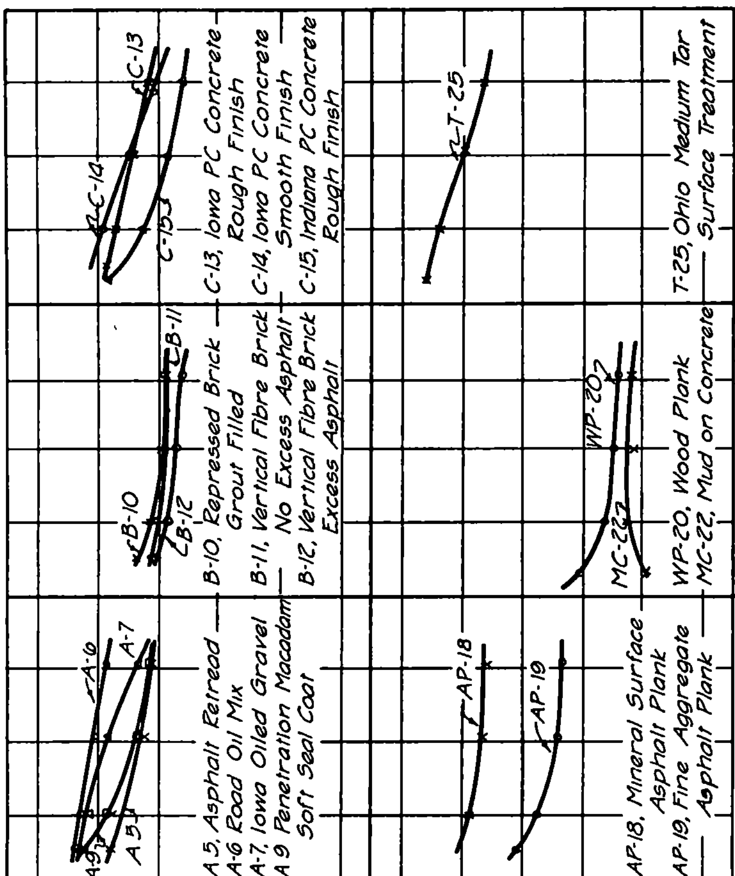


Figure 9b. Same Data as 9a, New Tread Tires Skidding Sideways on Wet Surfaces

lower in the side skid tests than the coefficients for similar conditions of tests on the wet high type asphaltic pavements As in the case of the tests on concrete, the results of tests on the brick were quite con-

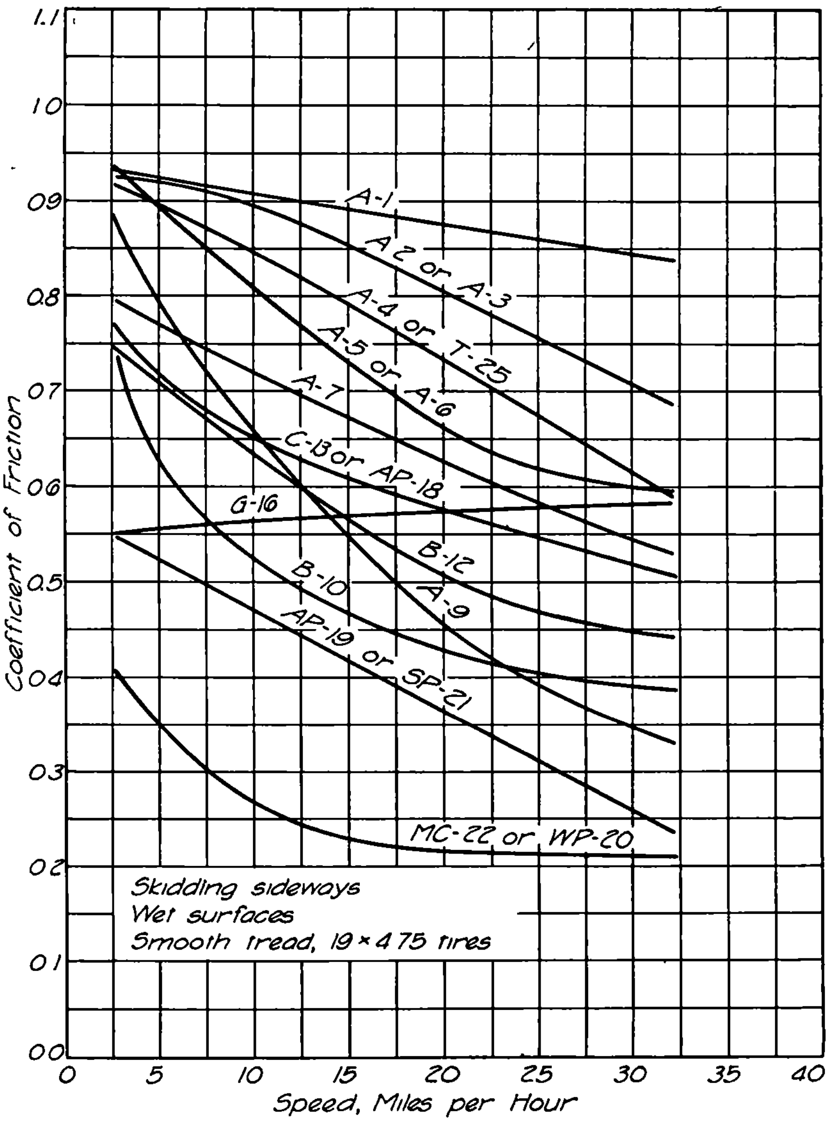


Figure 10a. Coefficients of Friction of Tires Skidding Sideways on Wet Surfaces. Smooth Tread Tires.

sistent It is of interest to note that the coefficients for the brick with excess filler were for the most part slightly higher than the coefficients for the grout filled brick and the vertical fibre brick with no excess filler

The joint filler used on the brick pavement with excess filler consisted

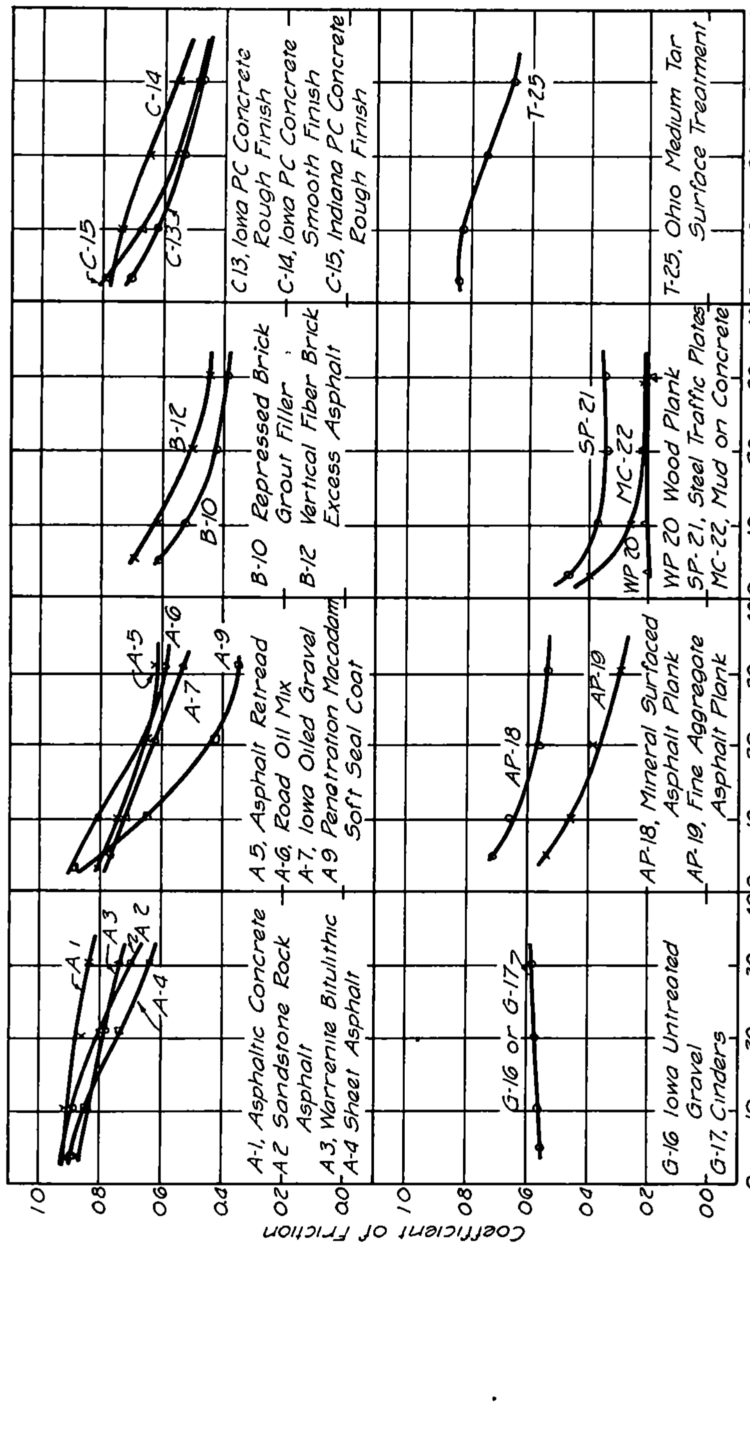


Figure 10b. Same Data as 10a. Smooth Tread Tires Skidding Sideways on Wet Surfaces

of a fairly hard asphalt with a penetration of 40 to 60 at 25° C, 100 g, 5 sec. Extraction tests indicated that the material on the surface contained about 45 per cent asphalt, 35 per cent sand, and 20 per cent

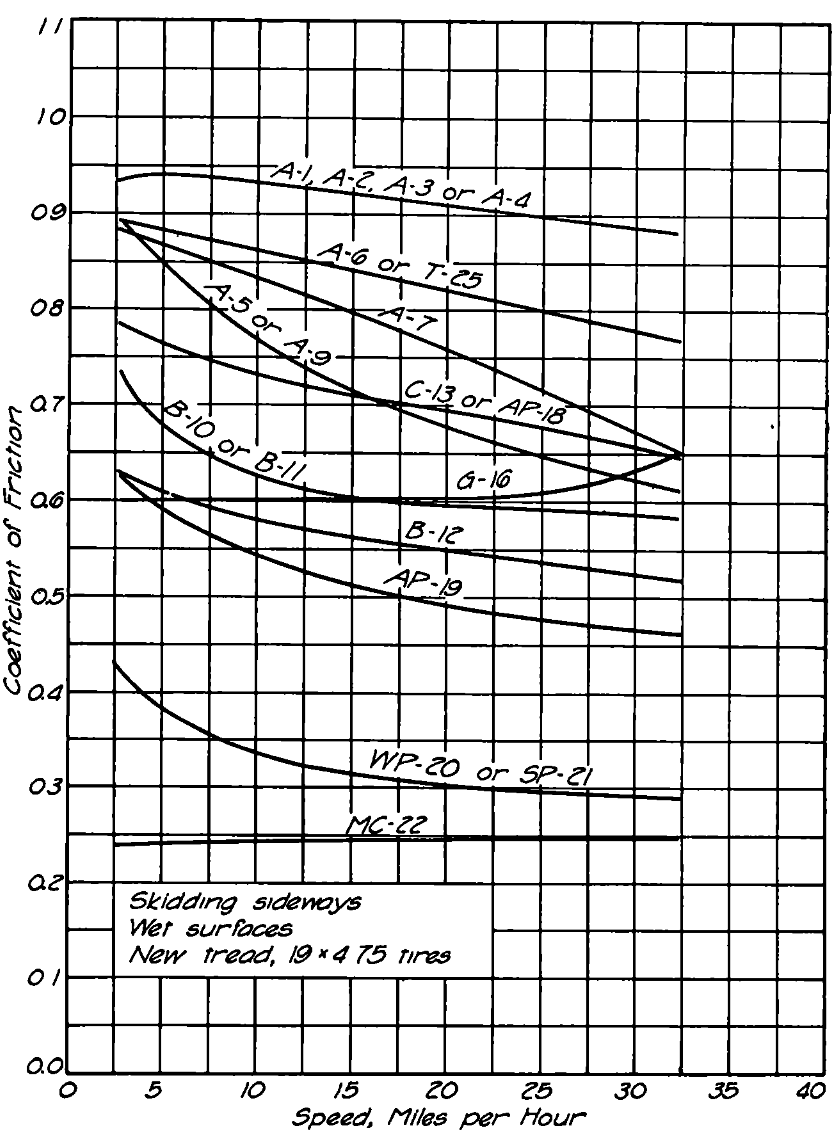


Figure 11a. Coefficients of Friction of Tires Skidding Straight Ahead on Wet Surfaces. New Tread Tires.

mineral filler passing a 200 mesh sieve Such a mixture has some of the characteristics of a sheet asphalt mixture, and since the coefficients for the sheet asphalt were consistently higher than those for the brick, provides a possible explanation as to why, in this particular case, the

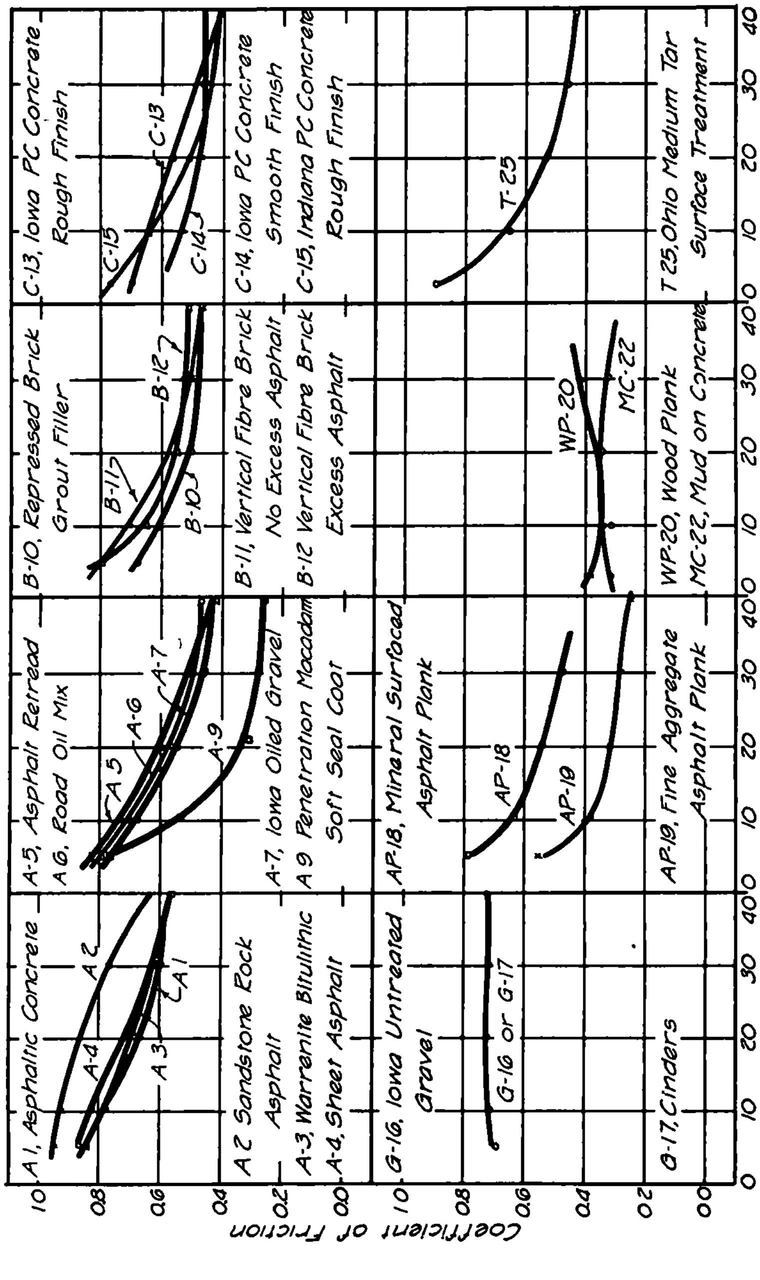


Figure 11b. Same data as 11a. New Tread Tires Skidding Straight Ahead on Wet Surfaces

brick with excess filler had a higher coefficient than the brick with no excess filler.

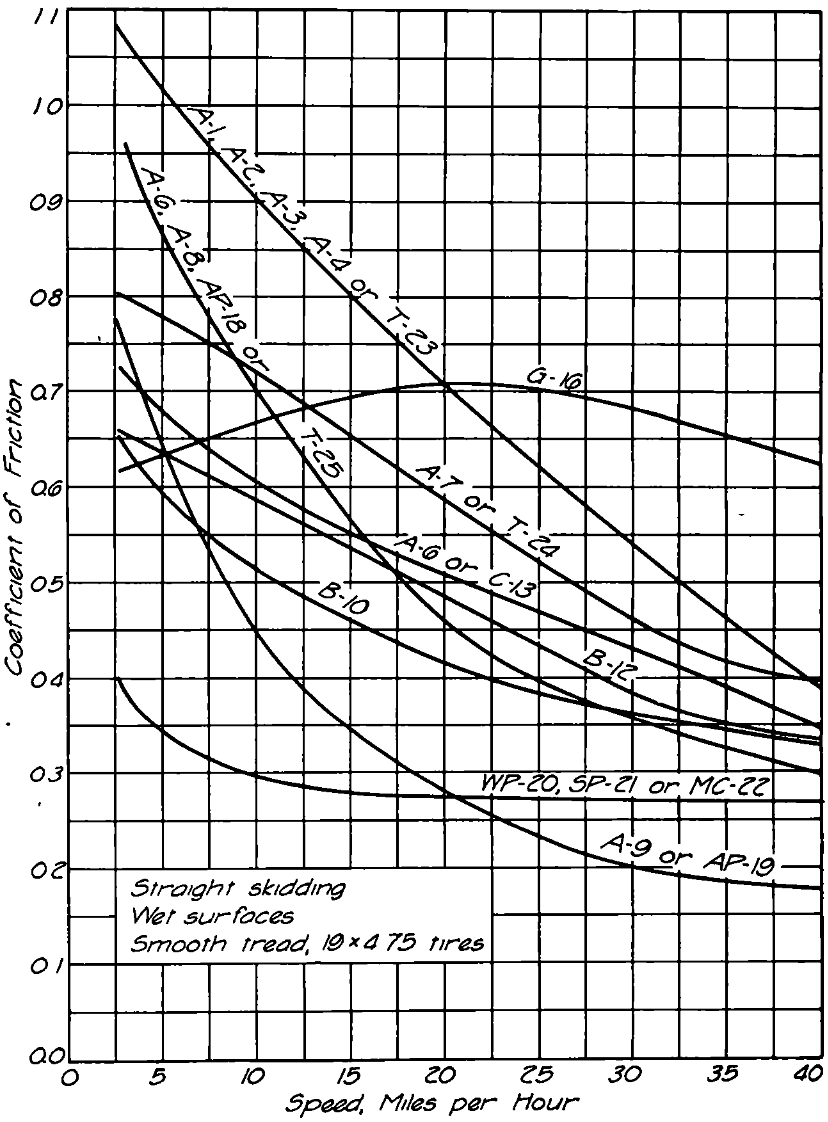


Figure 12a Coefficients of Friction of Tires Skidding Straight Ahead on Wet Surfaces Smooth Tread Tires.

Steel traffic plates and the hard wood plank bridge floor were practically as slippery when wet as Iowa mud on concrete It was evident when running tests on these surfaces that they were dangerous when

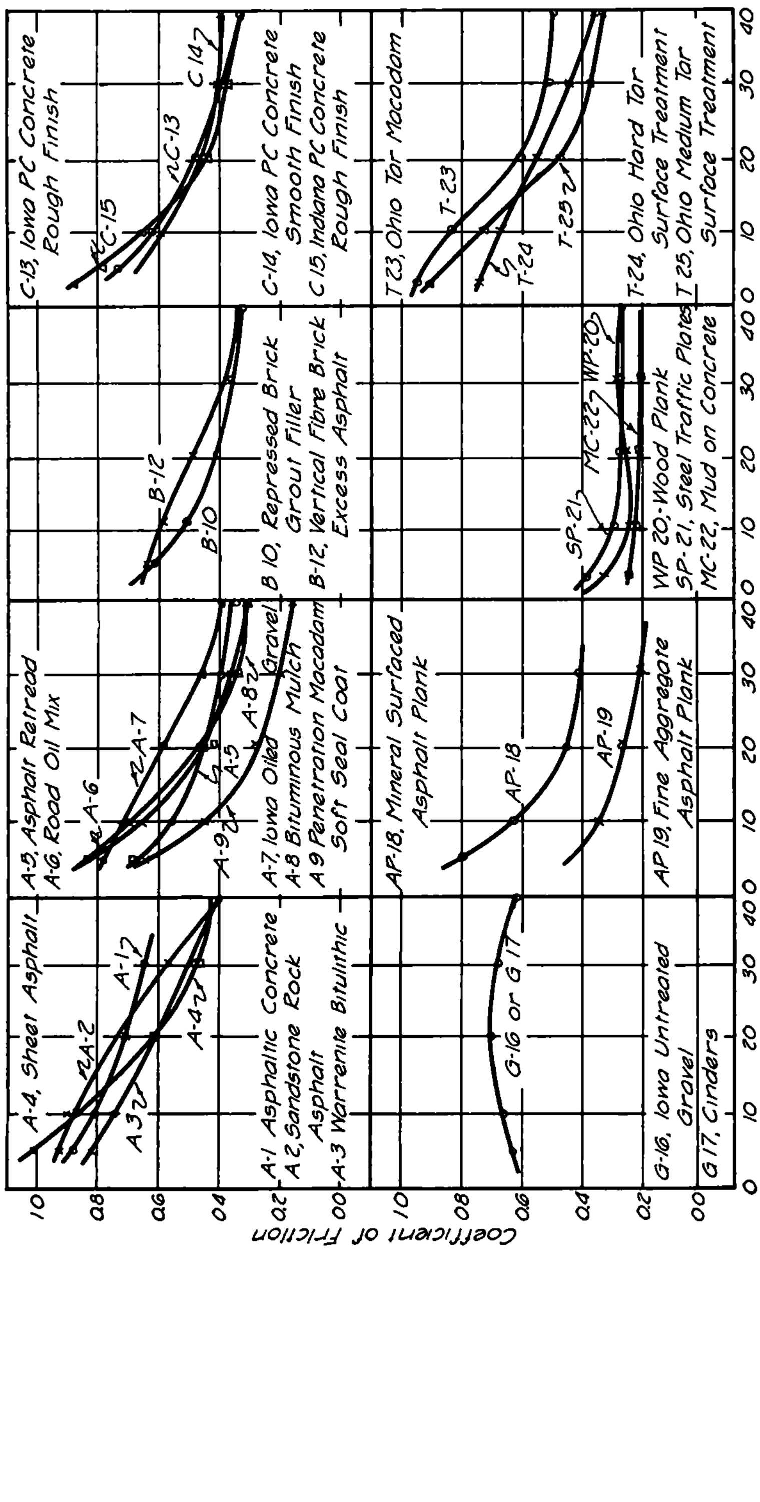


Figure 12b Same data as 12a. Smooth Tread Tires Skidding Straight Ahead on Wet Surfaces

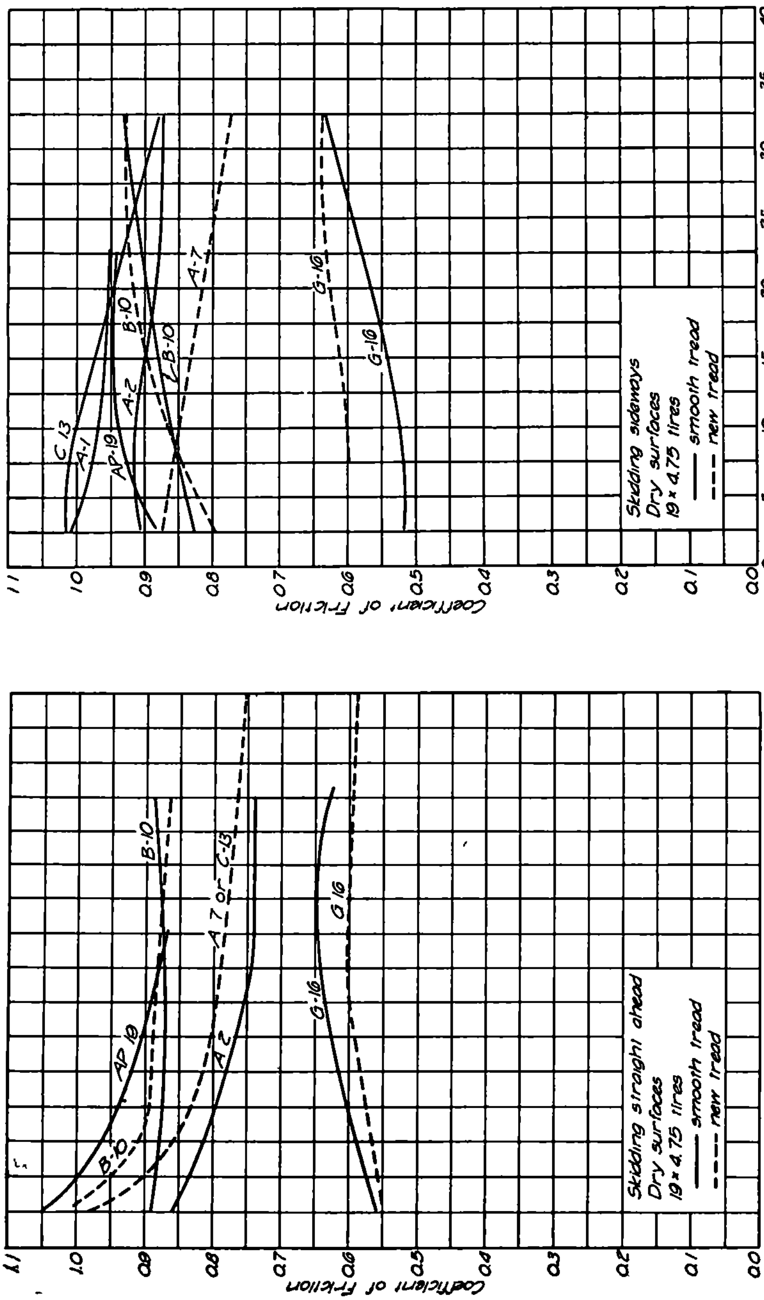


Figure 13a. Coefficients of Friction of Tires Skidding Straight Ahead and Side-ways on Dry Surfaces at Speeds Ranging From 3 to 30 m h.p

wet The steel traffic plates had a "non-skid" tread design on a solid steel plate It is possible that an open mesh design with fairly sharp edges would improve the skidding resistance of the steel plates Tests on wood plank floors covered with a tar or asphalt surface treatment indicated that this type of treatment will raise the coefficients to the same value obtained for similar surfaces on a macadam base

Mud on hard pavements is the cause of many accidents The placing of gravel, shale, cinders, or crushed rock on the shoulders and

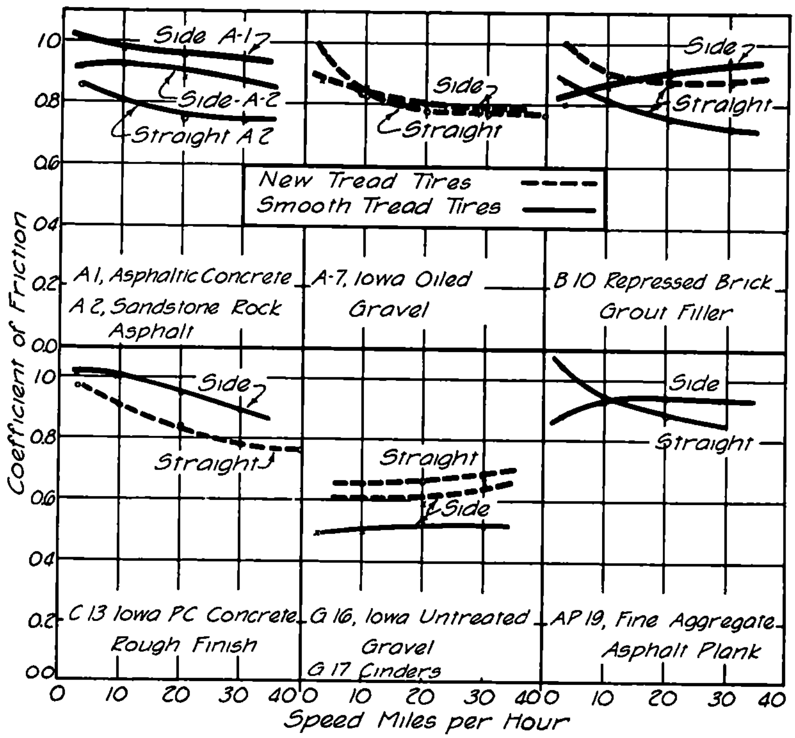


Figure 13b. Same Data as 13a. Skidding Straight Ahead and Sideways on Dry Surfaces.

at the approaches to all hard roads would not only correct a dangerous skidding condition, but would provide greater road widths for use in emergency passing on shoulders

The curves in Figures 13a and b show that the coefficients of friction for dry surfaces are considerably higher than for the same surfaces when wet, except in the case of gravel and cinders which were about the same wet or dry As with the wet surfaces the coefficients for a large number of dry surfaces decreased with an increase in speed, although the decrease was not as marked on the dry surfaces as when the same surfaces were wet An increase in the coefficient with an increase in speed was observed for the gravel, brick, and asphalt plank surfaces

As was to be expected, the coefficients of friction on sleet, ice, and snow covered surfaces, with and without tire chains, as shown in Figure 14, were low The tire chains used in these tests were of the stand-

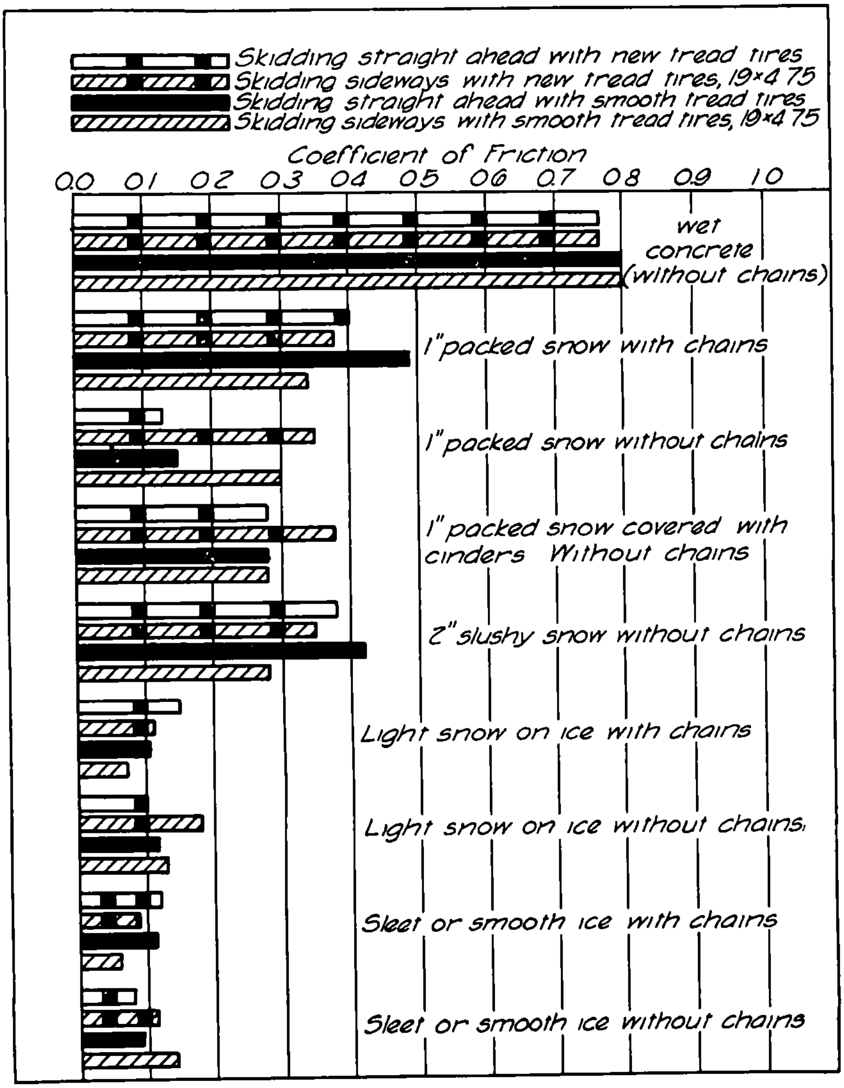


Figure 14 Relation between Coefficients of Friction of Old and New Tires with and without Tire Chains on Snow, Ice or Sleet, and Wet Concrete. All tests run at a uniform speed of 5 m.p.h.

ard four-link type with a spacing of about 6 inches between cross chains. For this spacing between cross chains, the tires were practically continuously in contact with the surface and little opportunity was provided to cause the chains to cut into the ice due to the weight of the car The results of the tests indicated that while there was a definite increase in the coefficient of friction when skidding straight ahead, a decrease in the coefficient was observed when skidding sideways That is, the standard four-link chains improve traction straight ahead but offer little or no protection from skidding sideways on ice or sleet

(*NOTE* Since this paper was read tests have been conducted using two-link chains with a spacing of about 3 inches between cross chains For this type of chain, there are two cross chains under each tire practically continuously. Under these conditions the chains cut into the ice getting the full effect of the weight of the car, thereby raising the coefficients to values ranging from 0 30 to 0 40 A slight increase in the coefficient was observed with an increase in speed)

COMPARISON OF THE THREE FORMS OF SKIDDING

An examination of the curves in Fig 15 will show that at the higher speeds there was a marked increase in the coefficients when skidding sideways as compared to the coefficients when skidding straight ahead A similar increase was observed when skidding was impending as compared to skidding sideways This same general trend was indicated in practically all of the surfaces tested (Figs 11 and 12) A greater proportional decrease in coefficient was observed in the straight skid tests than in the side skid tests This difference may be attributed to the fact that since in the straight skid tests, the tire, sliding on the same tread section heats to the point where the rubber becomes soft and weak, and, therefore, it does not have the same resistance to skidding that it has in the side skid and skidding impending tests where the skidding is not confined to one spot on the tire tread This point is further supported by the fact that at low speeds the coefficients for straight skidding are frequently higher than the coefficients for skidding sideways and for impending skidding

The relationship between the straight skid coefficients and the static, or skidding impending coefficients, is best illustrated by an analysis of the results in the slippage tests (Fig 16) In these tests on wet concrete it was found that the coefficient reached a maximum with a slippage of about 18 per cent and then decreased gradually to the value for 100 per cent slippage, or continuous sliding It is interesting to note that with five per cent slippage, a coefficient of about 90 per cent of the maximum can be obtained In other words with five per cent slippage a braking force equal to 90 per cent of the maximum braking force possible can be obtained on wet concrete On wet and dry gravel, the action was found to be quite different, and the highest coefficients were obtained with tires skidding continuously This can be explained by the fact that the tires plowed into the surface when the wheels were locked and gripped the road more firmly than when the

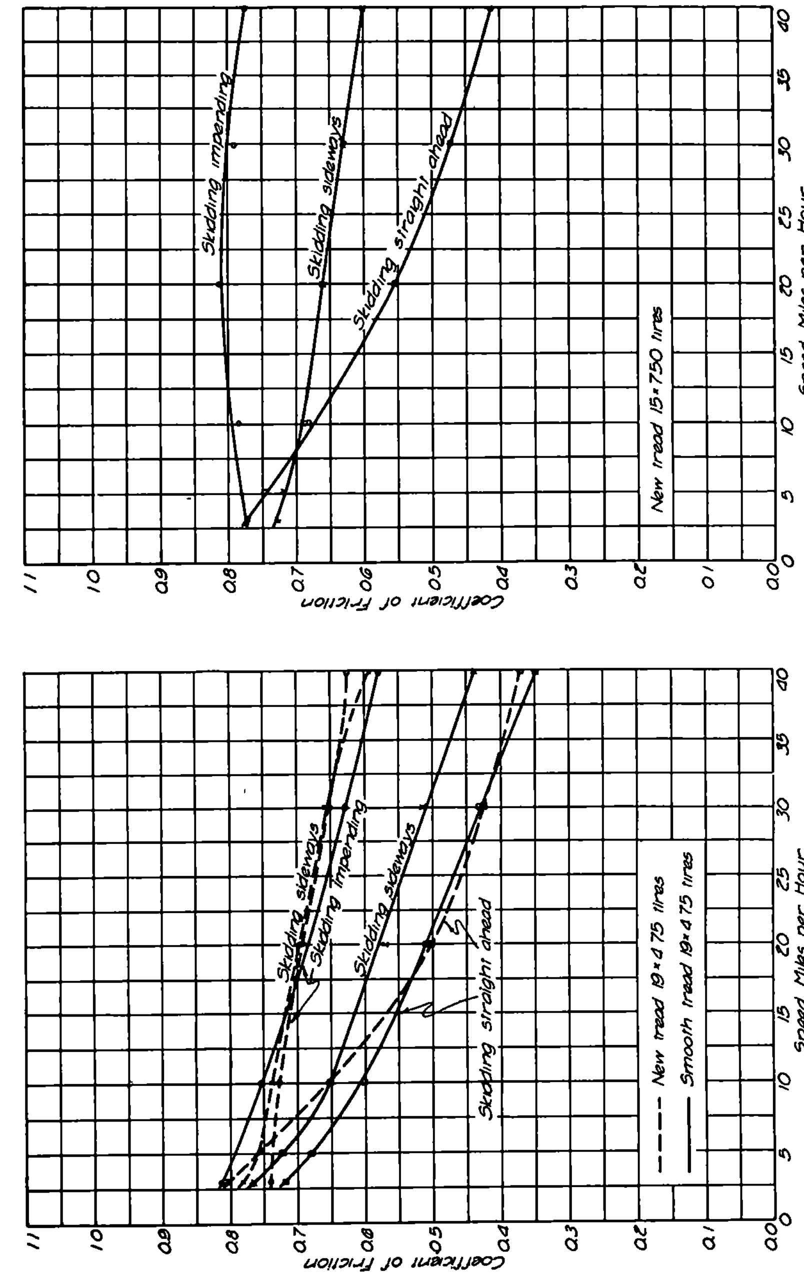


Figure 15. Relation between Static, Side Skid, and Straight Skid Coefficients of Friction on Wet Iowa Oiled Gravel (19 x 4.75 Tires) and Wet Concrete (15 x 7.50 Tires)

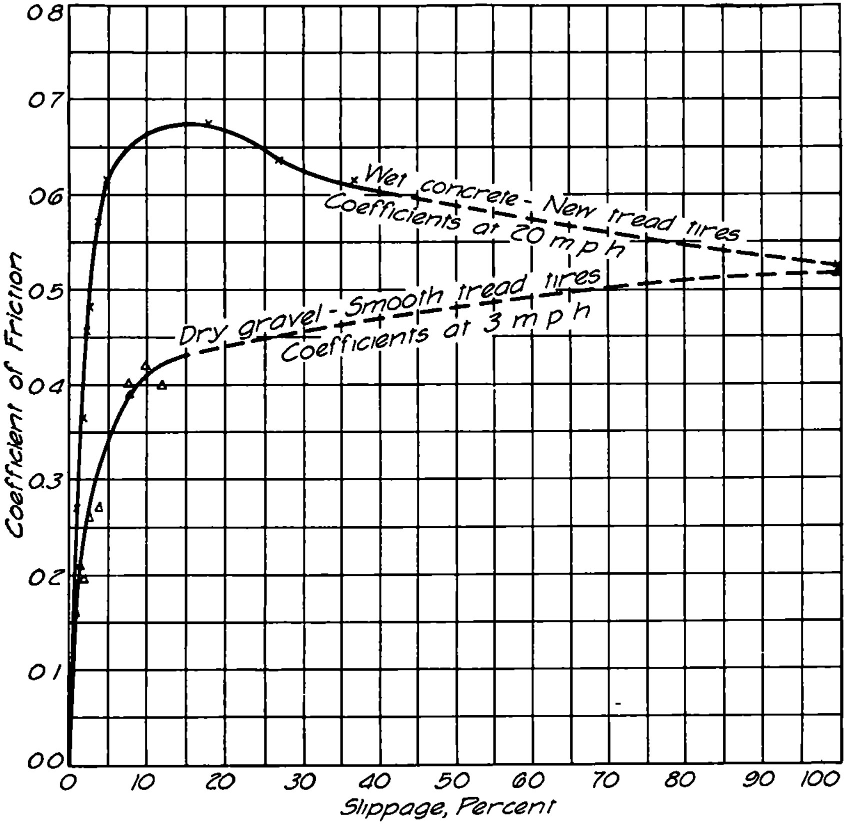


Fig. 16

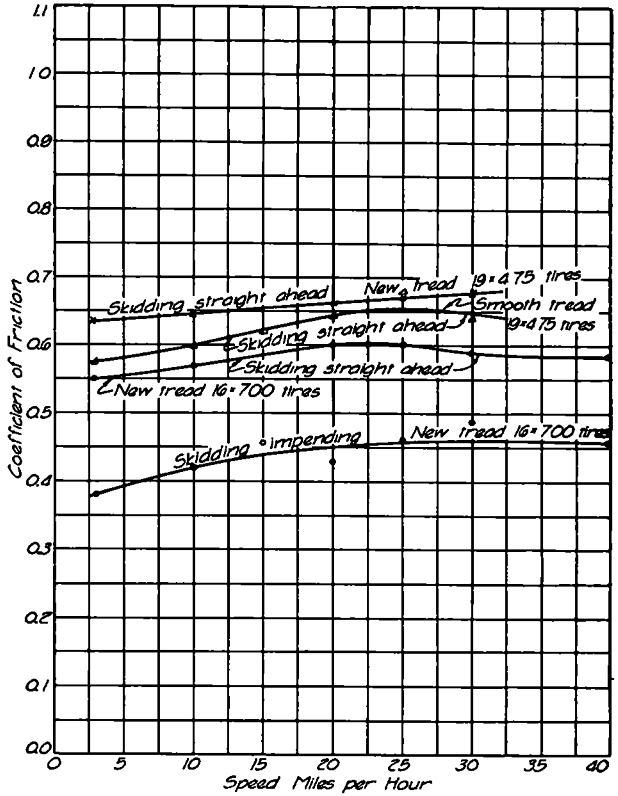


Fig. 17

Figure 16. Relation between Coefficient of Friction and Tire Slippage Caused by a Gradual Increase in the Braking Force

Figure 17. Relation between Static and Straight Skid Coefficients of Friction on Dry Untreated Gravel

wheels were turning This characteristic of gravel is further indicated in Fig 17 which shows the relationship between the static and straight skid coefficients of friction on dry gravel

VARIABLE FACTORS AFFECTING THE COEFFICIENT OF FRICTION

One of the most important phases of this investigation was the determination of the effects of the many known variables on the coeffi-

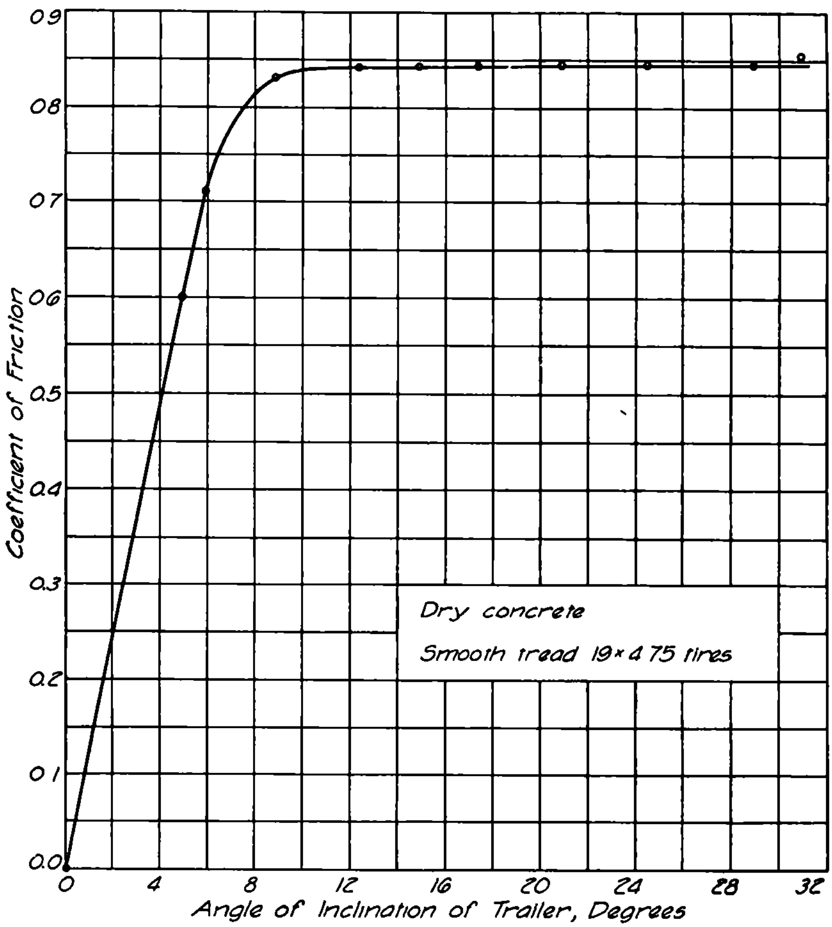


Figure 18. Effect of Angle of Inclination of Trailer on the Coefficient of Friction

cients of friction In making tests on the various surfaces, it was necessary to keep the conditions of test constant so that a fair comparison between the skidding characteristics of the surfaces could be made Considerable care was taken to determine the effects of the variables and to arrive at reasonable standards to be used in the comparison of road surfaces At least ten variable factors which affect the coefficient of friction were considered in this investigation They were, the form

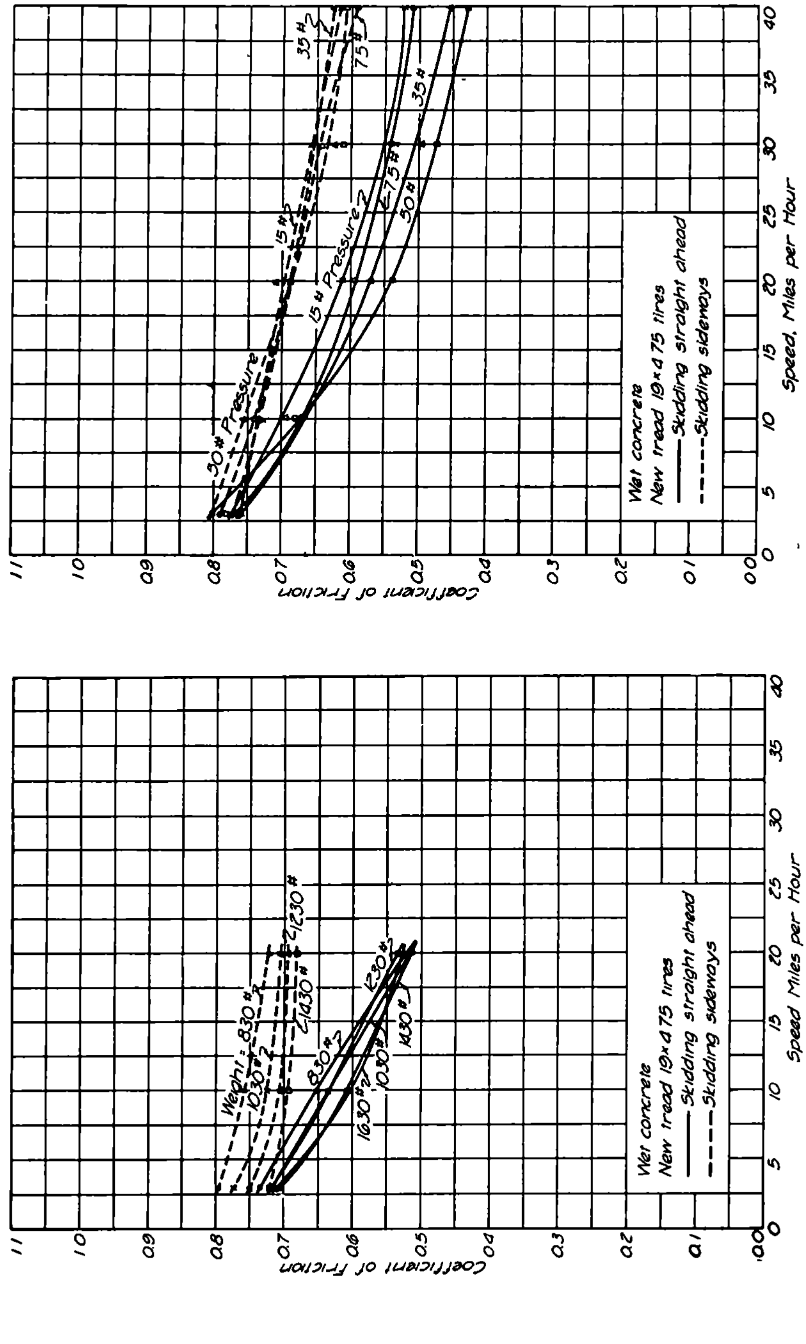


Figure 19. Effect of Variation in Gross Weight and Tire Pressure on Side Skid and Straight Skid Coefficients of Friction

of skidding, that is straight skidding, side skidding, or skidding impending, speed, varying the angle of inclination of trailer in the side skid tests, load, tire pressure, condition and type of tire tread, moisture on the road surface, temperatures, cleanness of the surface, and the condition of the brakes of motor vehicles

The first two of these variables have been discussed quite fully in the comparison of road surfaces A brief discussion of the effects of the other variables follows

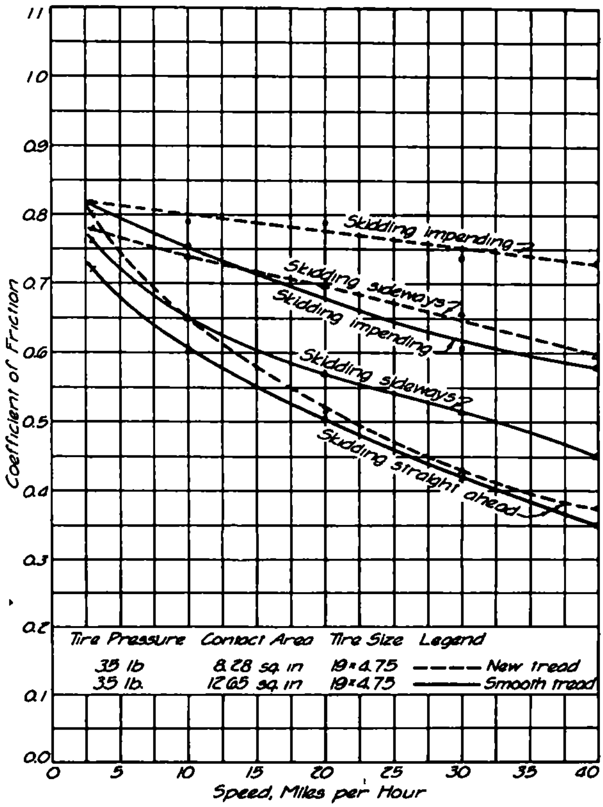


Figure 20 Effect of Tire Size and Tread Design on Coefficient of Friction New Tread and Smooth Tread Standard Balloon Tires. Tests on Wet Portland Cement Concrete.

Effect of varying the angle of inclination of the trailer The angle of inclination of the trailer was varied from 5 degrees to 30 degrees with the line of travel in the side skid tests This was accomplished by changing the length of the trailer tongue connection and of the dynamometer connections The tongue of the trailer was fastened to the truck with a swivel connection and was free to swing into any desired position Care was always taken to keep the dynamometer pulling in line with the axle of the trailer The results of the tests in angle variation (Fig 18) indicated that the coefficient increased rapidly up to an angle of about 12 degrees where it reached a maximum and remained constant at 30 degrees, the largest angle used in these tests

Effect of weight variation The effect of weight variation on the coefficient of friction was investigated for both straight and side skidding The total weight of the trailer was changed in 200-pound increments One series was run for each load as the increments of load were

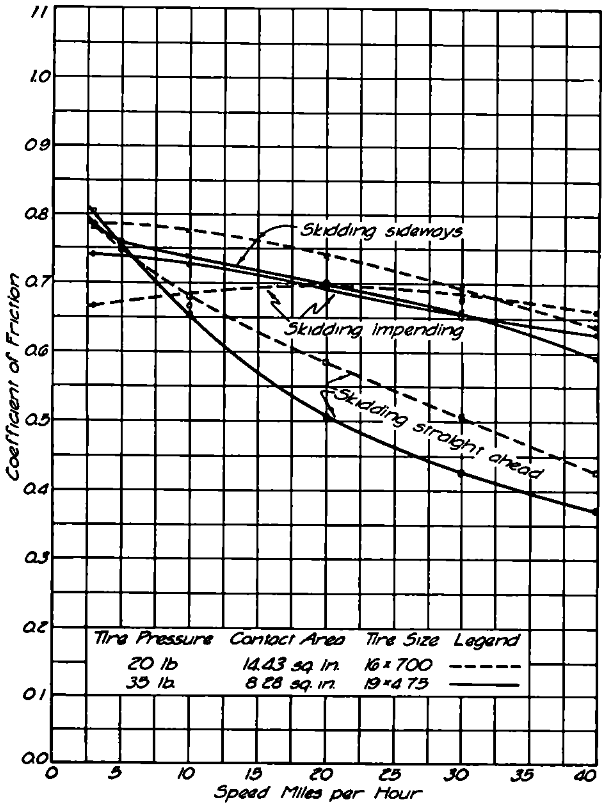


Figure 21 Effect of Tire Size and Tread Design on Coefficient of Friction New Tread Low Pressure and New Tread Standard Balloon Tires. Tests on Wet Portland Cement Concrete

added and check runs were taken as the weights were removed The results of these tests (Fig 19) indicate that a slight decrease in the coefficient accompanied an increase in weight The small increase in tire contact area accompanying the increase in load may have been responsible for the higher coefficients obtained with the lighter loads

Effect of tire pressure The effect due to variation in tire pressure was very similar to the effect due to weight variation That is, there was practically no difference in coefficient observed in the side skid tests and only a slight decrease in coefficients with an increase in pressure for the straight skid tests (Fig 19) The same explanation applies in this case as that offered in the case of the weight variation tests

Effect of condition of tires and type of tire tread Tests were run with smooth tread and new tread tires, and with low pressure balloon and standard balloon tires In the case of both the standard and the low pressure balloon tires, two distinct types of tread were tested, the

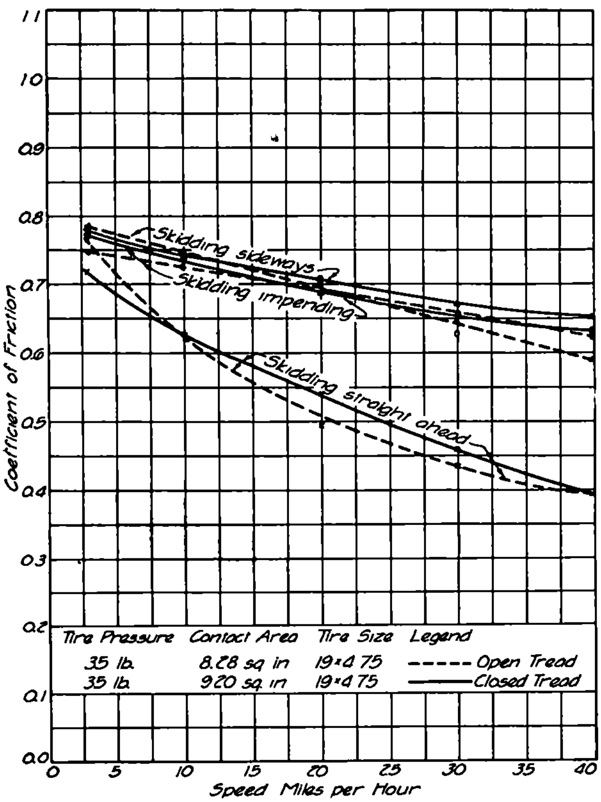


Figure 22 Effect of Tire Size and Tread Design upon Coefficient of Friction New Open and New Closed Type Tread Standard Balloon Tires. Tests on Wet Portland Cement Concrete.

open type tread and the closed type tread For the same test conditions the tire with the closed type tread had a greater contact area than the open type tread

The test results showed that the coefficients for the smooth tread tires were consistently lower than for the new tread tires for the three forms of skidding (Figs 20, 21, 22) This was true even though the contact area of the smooth tread tires was about 50 per cent greater than that of the new tread tires A possible explanation for the higher resistance of the new tread tires lies in the fact that the new tread tires provide more edges which can grip the surface and provide greater mechanical resistance than is possible with the smooth tread tires

However, when both tires had new treads, the results indicated that the contact area was a deciding factor in the coefficient of friction This appeared clearly to be the case when comparing the 19 by 4 75 inch and the 16 by 7 00 inch tires with the same tread design, since the coefficients were decidedly higher in the larger tire (Fig. 21) The coefficient for straight skidding for the 19 by 4 75 inch tire was lower because the contact area of this tire was only about one-half that of the larger tire, causing the tire to heat more when sliding on the small area, therefore, offering less resistance to sliding

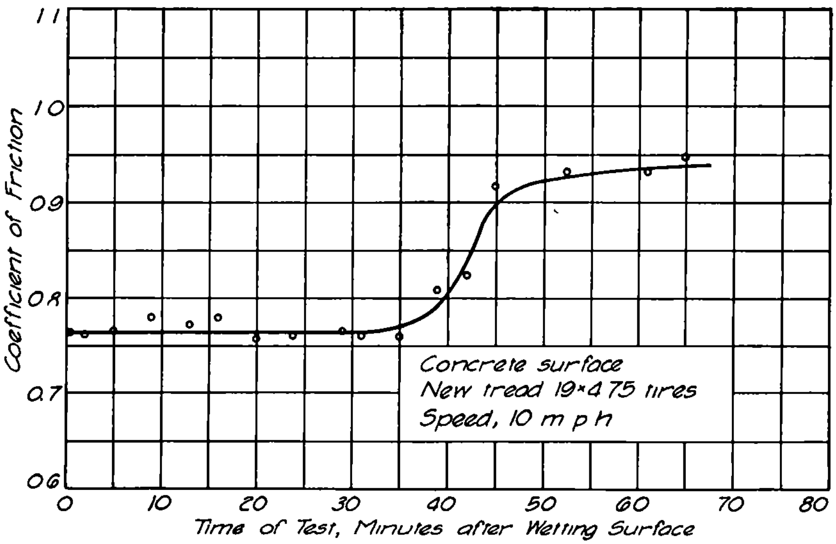


Figure 23 Effect on the Coefficient of Friction of Varying Amounts of Moisture on the Road Surface

It is interesting to note that the coefficients for the two types of standard balloon tires were very nearly the same in all of the tests (Fig 22), the small additional contact area in the closed type tread having had no noticeable effect

Effect of moisture The difference in the coefficients between the dry surfaces and wet surfaces was very marked and was discussed in connection with the tests on wet surfaces It should be stated here that the decrease in the coefficients on wet surfaces especially at the higher speeds was no doubt due to the lubricating action of the water on the surface At the lower speeds there was greater possibility for squeegee action of the tire on the surface, thus providing more intimate contact With an increase in speed, the tires fairly skimmed over the surface.

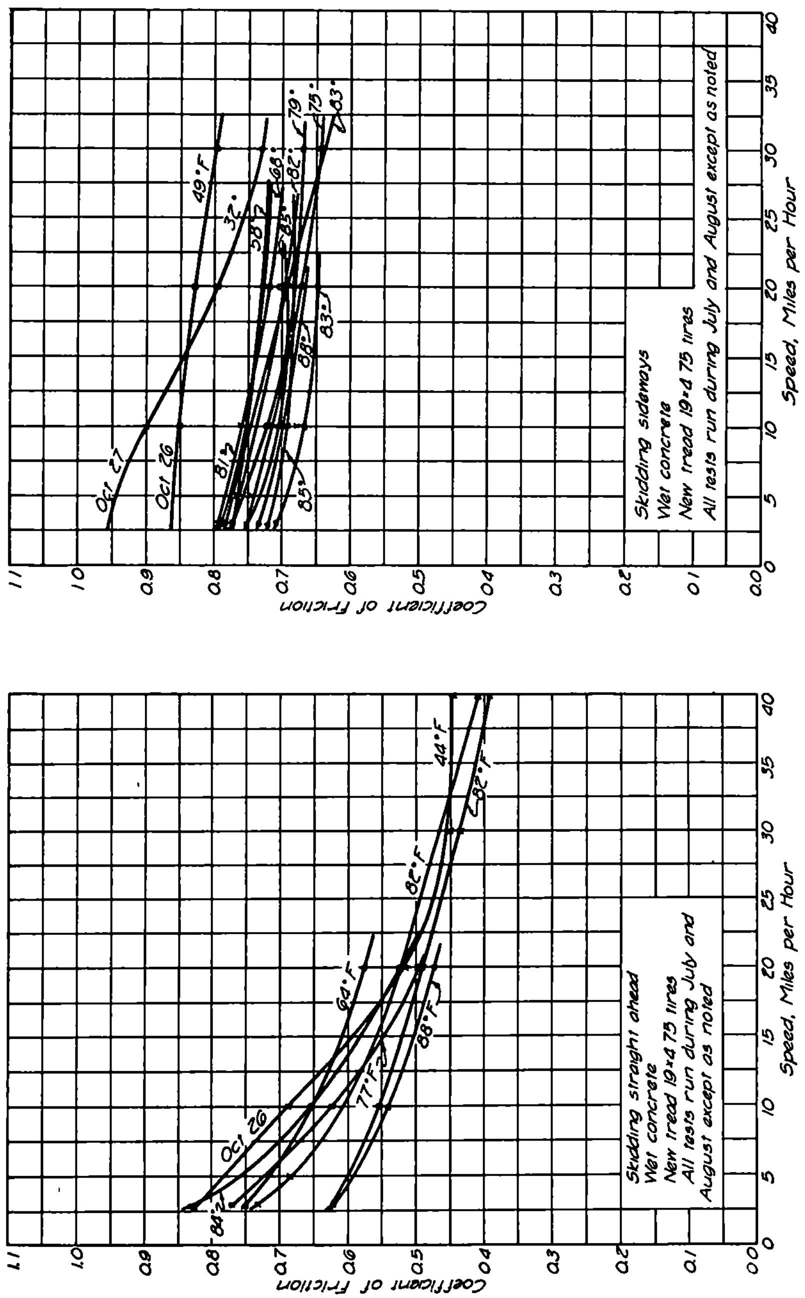


Figure 24. Effect of Change in Air Temperature on the Coefficient of Friction

There was some question as to the effect of variation in the amount of moisture on the surface To determine this effect the surface was thoroughly sprinkled before starting and tests were then run at frequent intervals without further sprinkling The results obtained (Fig 23), indicated that a period of 30 minutes could elapse after the surface was sprinkled before a definite increase in the coefficient was observed In some preliminary tests a fire hose was used to wet the surface There is some objection to this method since it is very likely that the surface is washed cleaner than would be the case during the average rain Care was taken in wetting the surface with the sprinkler to obtain the same surface condition that would exist during a rain

Effect of temperature Records of air temperature were kept during all of the tests Road surface temperatures and tire temperatures were measured in a large number of tests A definite increase in the coefficient of friction was observed with a decrease in air temperature (Fig 24). This difference was more marked in the side skid and static coefficients than in the straight skid coefficients This can again be explained by the effect of the heating of the rubber at the higher temperatures In the straight skid tests the temperature of the hot spot developed during sliding was not affected very much by changes in air temperature, hence the change in coefficient of friction was not noticeable

Effect of cleanness of surface An extreme case in the determination of the effect of cleanness of surface was that of mud on concrete The coefficients for this surface condition were all close to 0 2 for practically all conditions of test

Effect of condition of brakes of motor vehicles on the coefficient of friction The static (skidding impending) coefficient of friction and the straight skid coefficient determine the maximum braking force possible for these two conditions of skidding on a given road surface. If the brakes of the car can provide a braking force greater than the skidding resistance of the surface, it is quite likely that in an emergency stop one or more wheels will be locked and the tires caused to slide This is frequently the case on ice, snow, gravel, and cinders However, the results of tests conducted by the author on the condition of the brakes of 2,134 cars in 1932 (Fig 25) indicated that the braking effort of the average four-wheel brake car was only 51 5 per cent of the weight of the car including driver and passengers In other words, the average maximum braking force of the cars tested was considerably lower than the average resistance to stopping which the higher type road surfaces could provide, especially in the dry condition

The 51 5 per cent braking effort, when expressed in terms of the coefficient of friction for tires on road surfaces, is equivalent to a coefficient of 0 515 When the braking effort of a car is as low as this, it is not possible to slide any of the tires on high type surfaces unless the brakes are out of adjustment Tests of the cars having four-wheel

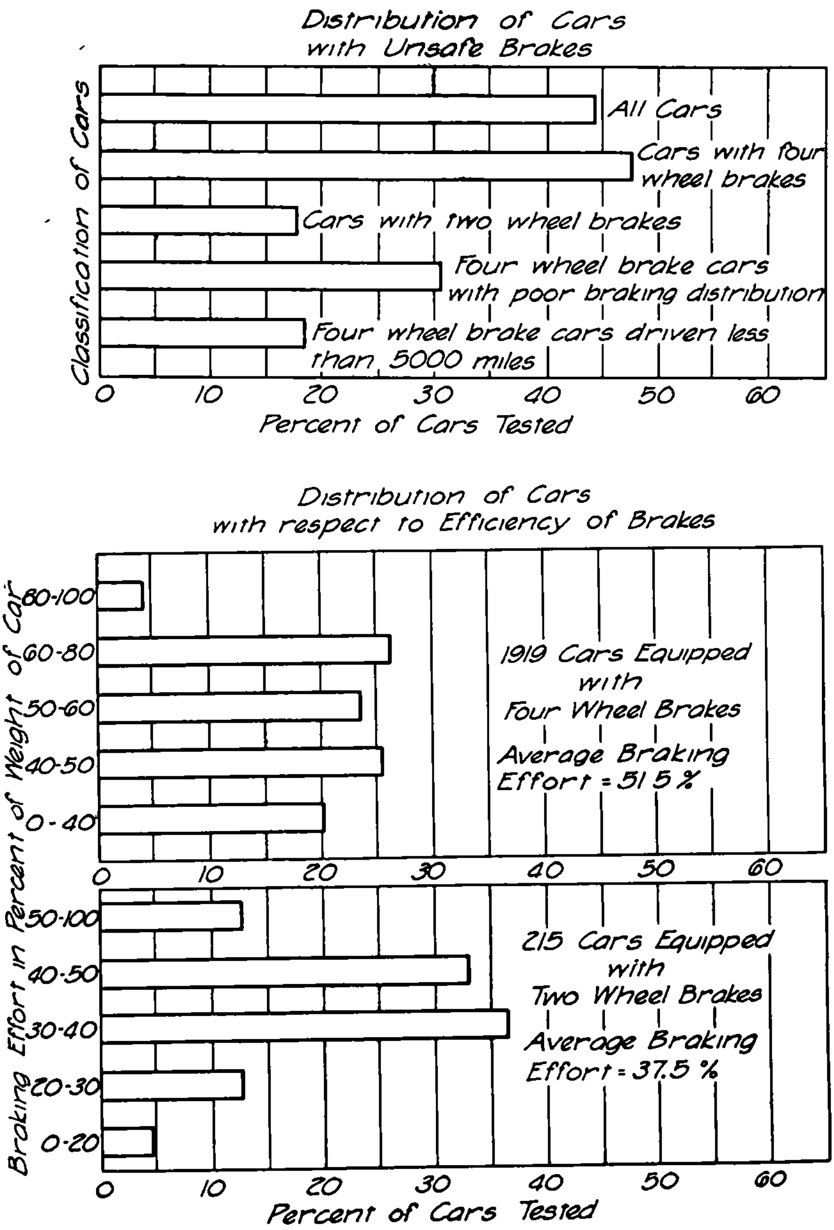


Figure 25. Condition of Brakes of 2,134 Cars Tested in 1932

brakes showed that this was frequently the case, since 31 per cent of the cars had unsatisfactory braking distribution, that is, greater than 40 per cent difference between the left and right wheels, either front or back It is evident from these tests that poor braking distribution is responsible for many of the skidding accidents, especially those on wet surfaces

THEORY OF SKIDDING

The following theory of skidding is offered as a possible explanation of the reasonableness of the results obtained in the weight variation, pressure variation, and similar tests The total frictional resistance to skidding is equal to the true frictional resistance between the two surfaces, plus the resistance that results from a mechanical interlocking of the tire tread with the road surface This mechanical resistance is brought about by the small particles of rubber interlocking with the road surface and shearing off as the tire slides forward The true frictional resistance is theoretically proportional to the normal force between the two surfaces The mechanical resistance, however, is dependent not only on the normal force but also on the area of contact of the tire

The contact areas of the tires increase as the weight is increased An examination of the records of the tire imprints (Figs 26 and 27) shows that the unit pressure between the tires and the roadway surface was nearly the same for all the loads tested It is to be expected, therefore, that the coefficient for straight skidding should remain fairly constant as the weight is increased and it was found by test that this was the case

Conditions are different in the side skid tests The increase in load resulted in a greater deflection of the tire laterally in addition to the increase in contact area It is possible that the greater deflection of the tire with an increase in load changed the nature of the contact of the tire with the road surface and thus accounted for the slight decrease in the side skid coefficients

On wet surfaces the squeegee action of the edges in the tread design of the new tires provides a more intimate contact of the tire with the road surface, which reduces the lubricating effect of the water, and thereby increases both the true frictional and the mechanical resistance for the two materials

At the higher speeds, the coefficients for new tread tires were equal to or greater than the coefficients for smooth tread tires although the contact areas of the smooth tread tires were approximately twice as large as the areas of the new tread tires This result is in conformity with the theory set forth above That is, since there is very little squeegee action in the case of the smooth tread tires, the same intimate contact with the road surface is not provided by the smooth tread tires as is provided by the new tread tires, and, accordingly the frictional resistance of the smooth tread tires should be less

Frictional Resistance Necessary for Protection against Skidding

In the following analysis, limiting coefficients of friction which will assure reasonable protection against skidding are derived.

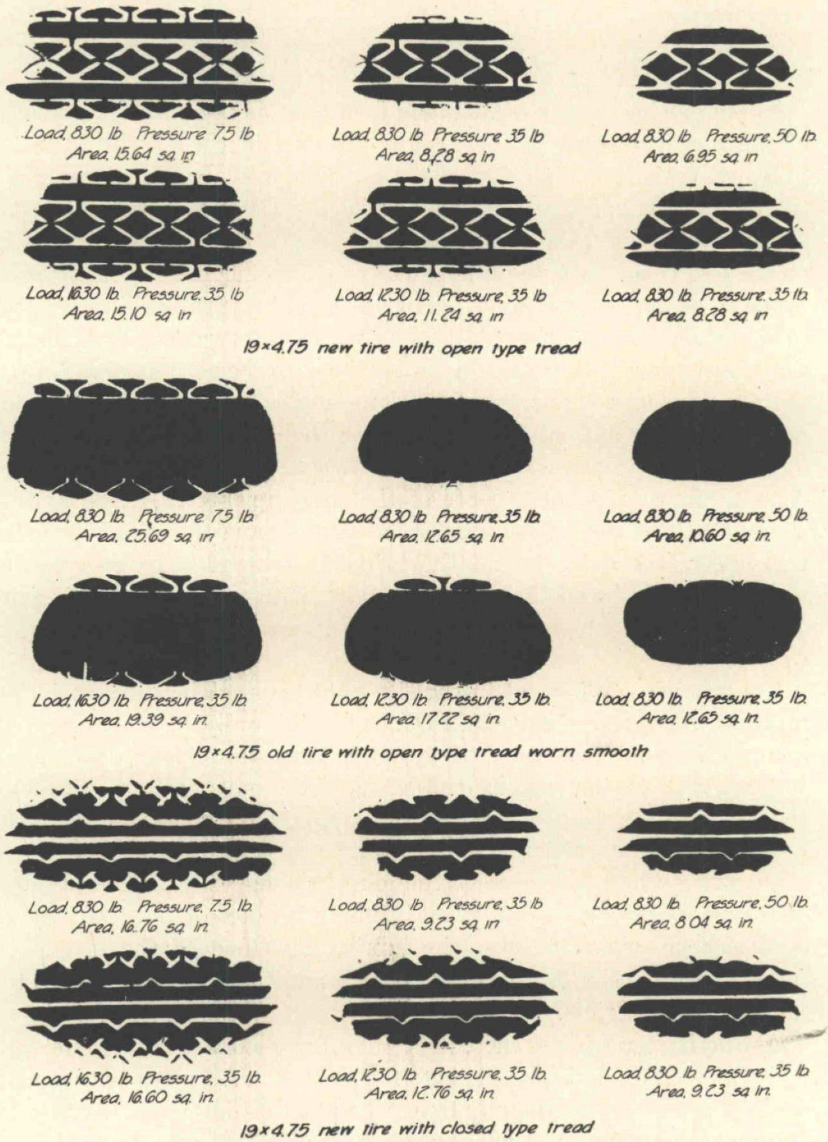


Figure 26. Imprints of Standard Balloon Tires Showing Effects of Load and Pressure Variations on Contact Areas. The load given under each imprint is the total trailer load (two wheels).

It is possible to cause a car to skid on any road surface, but it is recognized that the careful driver will be safe from skidding except on slippery surfaces. The question then arises, what constitutes a slippery surface?

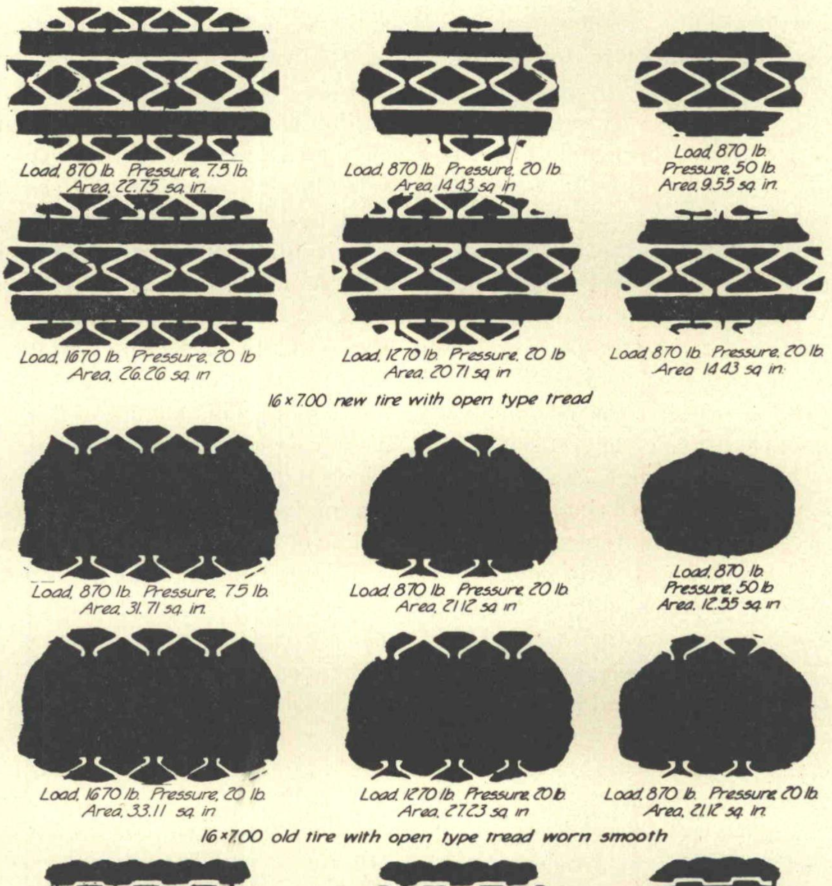


Figure 27. Imprints of Low Pressure Balloon Tires Showing Effect of Load and Pressure Variations on Contract Areas. The load given under each imprint is the total trailer load (two wheels).

In driving a car on a curve, the tires and road surface must provide frictional resistance for two distinct purposes, to oppose the centrifugal force which acts normal to the path of the car, and to resist either the driving force or the retarding force in the line along which the wheels of the car tend to travel. The force in the line of travel is subject to great variation on the rear wheels since they usually furnish the driving and accelerating force for the car, and also deliver a large retarding force when the brakes are applied. The force in the line of travel on the front wheels is only subject to the variation caused by the application of the front wheel brakes, except when the car has front wheel drive. The frictional force in the line of travel is relatively high on sharp curves because of the large slip angles of both front and rear wheels. The slip angle is the angle between the horizontal diameter of the wheel and the tangent to the path along which the wheel travels.

If the driving force on the rear wheel is held equal to the sum of the frictional resistance due to the slip angle and the tractive resistance of the car, the frictional force in the line of travel on the rear wheels (driving wheels) can be practically eliminated. This condition is not possible for the front wheels, except in the case of front wheel drive.

In driving from a sharp curve onto a long radius curve the slip angles decrease, thereby bringing about a reduction in the frictional resistance necessary to counteract the force in the line of travel, and permitting a higher safe speed of the car. However, at higher speed, there is greater air resistance which in part makes up for the loss in frictional resistance caused by the reduction of the slip angles.

Therefore, although the frictional resistance at the front wheels in the line of travel on a long radius curve should be less than that on a short radius curve, it can still be very large at the rear (driving) wheels due either to a large accelerating or braking force and thus can contribute to skidding.

Highway engineers have not been considering the frictional force in the line of travel in designing highway curves but have based their designs for curves on the requirements for centrifugal force only. The frictional force which should serve as a basis for the design of curves on a given highway is the *resultant* of the frictional force which acts normal to the path of the car and the frictional force which acts in the line along which the wheels tend to travel. Tests to determine the possible range for the latter force have not been completed, but on the basis of the data now at hand, it appears that a side skid or static coefficient of friction of 0.5 or greater at a speed of 30 miles per hour (or 0.6 at 20 miles per hour) is desirable, in order that a useful coefficient of at least 0.3 to 0.4 will be available to counteract centrifugal force. Thus, a wet surface with a static or side skid coefficient of 0.6 at 20 miles per hour should provide a coefficient of at least 0.4 to counteract centrifugal force and at the same time it should provide a coefficient of at least 0.45 to counteract the driving or retarding force in the line of travel of the wheels of the car $[(0.6)^2 = (0.4)^2 + (0.45)^2]$. Under these conditions it should be possible to turn a 50-foot radius street corner safely at 20 miles per hour under fairly severe driving conditions, such as increasing speed or moderate braking

On through highways a minimum radius of curvature of 1,000 feet with superelevation of one inch per foot, is standard in many states The maximum safe speed on such a curve on a road surface which can provide a useful coefficient of friction of 0 3 to counteract centrifugal force, is 75 miles per hour This should be provided by a surface with a static or side skid coefficient of 0 5 at 30 miles per hour (about 0 36 at 75 miles per hour), which should also provide a coefficient of 0 2 to counteract the driving or retarding force at that speed $[(0\ 36)^2 - (0\ 3)^2 + (0\ 2)^2]$ For through highways, therefore, surfaces with static or side skid coefficients of 0 5 or higher at 30 miles per hour may be considered reasonably safe

Skidding on curves can be prevented to a large extent by a careful driver since he generally receives and recognizes a fairly clear warning signal before skidding starts In driving on curves at a speed for which a coefficient of 0 1 was required to counteract centrifugal force, it was found that a blindfolded passenger could not sense clearly when the car was on the curve and when on the tangent By increasing the speed to the point where this coefficient was increased to 0 2, the passenger could clearly sense the curve and some discomfort was experienced But when the speed was increased to the point where a coefficient of 0 3 was necessary to counteract centrifugal force, a decided side pitch force was encountered both by the driver and the passenger This side pitch was distinctly uncomfortable and should serve as a warning to reduce speed

Safe Stopping Distance

The brake tests on the 1,919 four-wheel brake cars indicated that it should be possible to lock one or more wheels of 80 per cent of the cars on a surface with a straight skid coefficient of 0 4 or less With one or more wheels locked on a tangent, a side skid is likely to develop which may be harder to control than the side skid on a curve However, if a surface can provide an average straight skid coefficient of 0 5 over the entire range of speed in which the car is being stopped, it should be possible to stop a car with four-wheel brakes at a rate of 16 feet per second per second Although in an emergency it may be necessary to stop at such a rate or even faster, it should be recognized that this is a stopping rate close to the limit for the comfort and safety of the occupants of the car, since each occupant must resist being thrown forward by a force equal to one-half his weight While such a rate of stopping might be considered desirable on dry surfaces, many drivers would hesitate to stop at that rate on a wet surface It appears then that surfaces which can provide a straight skid coefficient of 0 4 or higher at 40 miles per hour when wet can be considered to be reasonably safe from skidding.

SUMMARY

The results of this investigation may be summarized as follows

1 A marked decrease in the coefficient of friction as speed of car increased was observed on all *wet* surfaces tested, except the gravel and cinder surfaces for which a slight increase in the coefficient was observed with an increase in speed

2 The same characteristic was observed on *dry* surfaces, although the decrease in coefficient was at a much lower rate and an increase in the coefficient with an increase in speed was observed not only on gravel but also on brick and asphalt plank surfaces

3 The coefficients of friction for the wet bituminous surfaces covered a wide range of values The coefficients for the wet high type asphaltic and tar surfaces were consistently higher than the coefficients for the other wet surfaces tested However, unusually low coefficients were also obtained on bituminous surfaces, especially on the wet penetration macadam with a soft seal coat

4 The coefficients of friction increased with an increase in the hardness of the asphalt or tar binder used in the bituminous surfaces

5 The coefficients for the wet rock asphalt surface with the "sandpaper" finish were higher than for the wet asphalt retread with the rough coarse textured "non-skid" surface

6 The coefficients for the wet portland cement concrete surfaces were 15 to 40 per cent lower than for the wet high type asphaltic surfaces The results for the concrete surfaces were remarkably consistent even though the finish was different in each of the three surfaces tested

7 The relative resistance to skidding for wet surfaces, starting with the surfaces with the highest resistance was as follows. high type asphaltic pavements, tar macadams, asphalt retread and oiled gravel, untreated gravel, portland cement concrete, mineral-surfaced asphalt plank, brick, asphalt penetration macadam with soft seal coat, fine aggregate type asphalt plank, steel traffic plates, hard wood plank, mud on concrete or other hard surface, and snow, sleet, and ice covered surfaces

8 The standard four link tire chains increased the skidding resistance of sleet covered surfaces when skidding straight ahead but reduced the resistance when skidding sideways (In recent tests using two link chains on ice, coefficients ranging from 0 30 to 0 40 were observed)

9 In the tests on wet surfaces at 30 miles an hour, the straight skid coefficient was 10 to 20 per cent lower than the side skid coefficient for the same surface Under similar conditions the static and side skid coefficients were approximately the same However, for the low pressure balloon tires, the static coefficients were 10 to 30 per cent higher than the side skid coefficients on the wet high type surfaces

10 On untreated gravel the static coefficients were about 10 per cent lower than the straight skid or side skid coefficients.

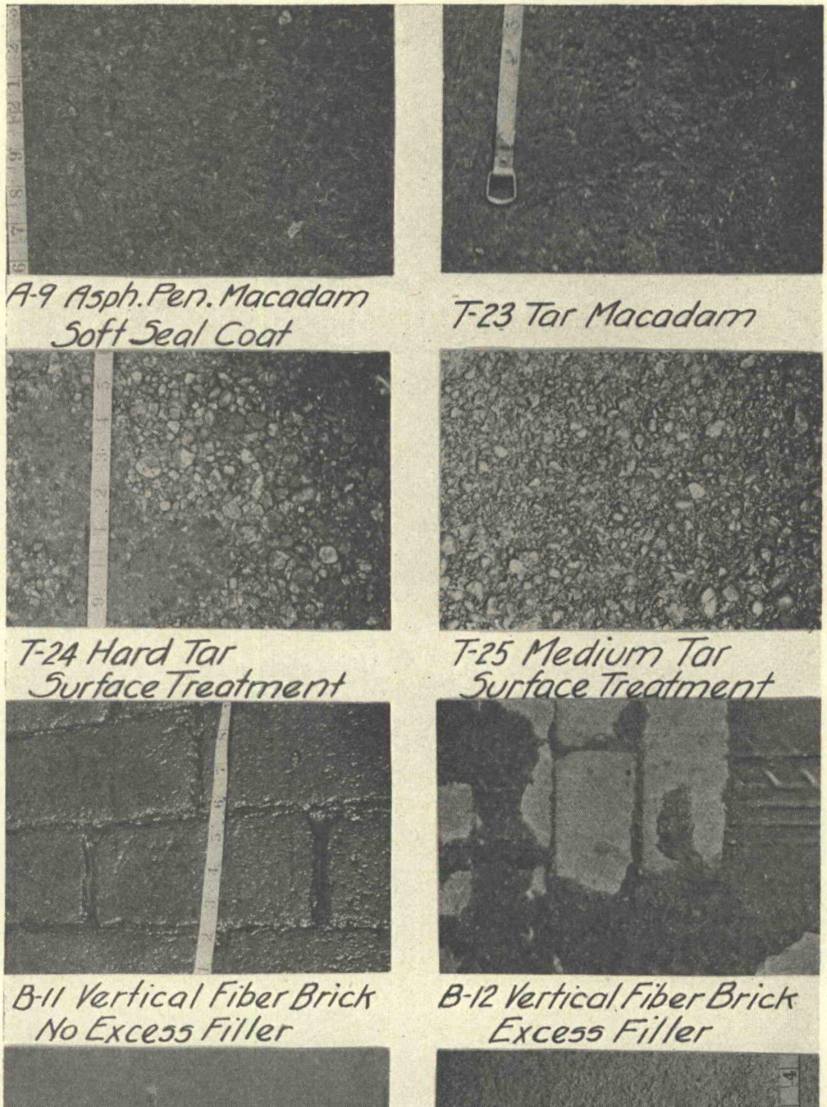


Figure 28. Surface Texture of Some of the Pavements Tested

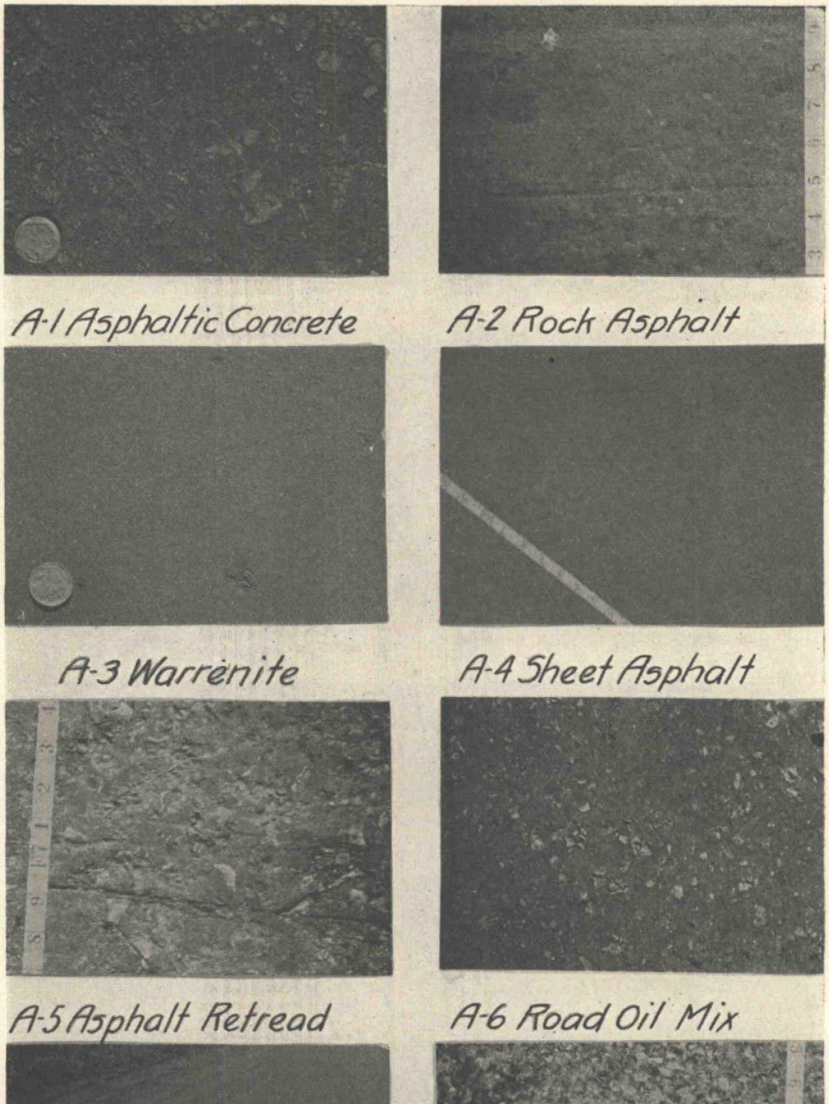


Figure 29. Surface Texture of Some of the Pavements Tested

11 In the slippage tests on wet concrete, it was found that the static coefficient reached its maximum value with a slippage of about 18 per cent and gradually decreased in value with an increase in slippage to the value for the straight skid coefficient at 100 per cent slippage With five per cent slippage a braking force equal to 90 per cent of the maximum braking force possible on wet concrete was obtained

12 A variation in the load on the tires or a variation in tire pressure had little effect on the coefficient of friction, only a slight decrease in the coefficients having been observed with an increase in load or in tire pressure

13 The same coefficients of friction were obtained for two standard balloon tires of the same size but with different tread designs

14 An increase of approximately ten per cent in the coefficients of friction was observed for the low pressure balloon tires when compared with standard balloon tires of the same make and with contact areas only sixty per cent as large

15 At the higher speeds, the coefficients for new tread tires were equal to or greater than the coefficients for smooth tread tires although the contact areas of the smooth tread tires were approximately twice as large as the areas of the new tread tires This result is in conformity with the theory set forth on page 39

16 A fairly definite increase in the coefficient of friction was observed with a decrease in temperature The hot spot on the tire which developed at the higher speeds when sliding straight ahead was largely responsible for the consistently lower coefficients observed in this form of skidding

17 The average maximum braking effort of 2,134 cars tested in 1932 was only 51 5 per cent of the weight of the car, which is equivalent to an average coefficient of friction of 0 515, indicating that a majority of these cars were not able to take advantage of the coefficients of friction which dry surfaces can provide

18 In driving a car on a curve, the tires and road surface must provide frictional resistance, (1) to oppose the centrifugal force which acts normal to the path of the car, and (2) to resist the driving or retarding force in the line along which the wheels of the car tend to travel

19 In the design of highway curves due allowance should be made for the frictional force in the line of travel in deciding upon the useful coefficient of friction to counteract centrifugal force normal to the line of travel.

CONCLUSIONS

1 An important factor in surfaces having high coefficients of friction is the presence of gritty particles which give the surface a "sand paper" texture Conversely, the glazing or polishing effect of traffic on certain surfaces is responsible in part for the decrease in the coefficients observed for these surfaces tested wet

2 The coefficients of friction at speeds of 3 to 5 miles per hour are not a true indication of the coefficients at higher speeds

3 The increase in coefficient obtained for the tire with the larger contact area but with the same tread design substantiates the theory that the total frictional resistance for tires on road surfaces is equal to the true frictional resistance for the two materials plus the mechanical resistance which increases with the contact area of the tire

4 On wet surfaces the squeegee action of the edges in the tread design of the new tires provides a more intimate contact of the tire with the road surface, which reduces the lubricating effect of the water, and thereby increases both the true frictional and the mechanical resistance for the two materials

5 To be reasonably free from the dangers of skidding, road surfaces when wet should have a static or side skid coefficient of 0 5 or higher at 30 miles per hour and a straight skid coefficient of 0 4 or higher at 40 miles per hour.

COEFFICIENT OF FRICTION BETWEEN TIRES AND ROAD SURFACES

By Karl W. Stinson, *Associate Professor of Automotive Engineering*, and Charles P. Roberts, *Assistant Professor of Mechanical Engineering, Ohio State University*

SYNOPSIS

Tests to determine the "Coefficient of Friction between Automobile Tires and Road Surfaces" were made using a two-wheel trailer connected to a towing car through a hydraulic dynamometer. All runs were made during rainfall, the test speeds were maintained throughout the application of the brake, and data were obtained to determine the rolling and sliding coefficients of friction. Smooth and nonskid tires were used and tests were made on portland cement concrete, brick and bituminous road surfaces at speeds of five to forty-five miles per hour.

Two important trends shown by these tests are: That the coefficients of friction on all road surfaces decrease with increase of speed, and that the difference between the rolling and sliding coefficients increases with increase of speed. These relations are very significant from the standpoint of safety.

In these tests the straight ahead sliding coefficients of friction between nonskid tread tires and the various wet road surfaces tested at thirty miles per hour ranged as follows: Bituminous concrete 0.54 and 0.42, sheet asphalt 0.41, portland cement concrete 0.41, vertical fiber wire cut brick whitewashed before application of filler 0.52, vertical fiber wire cut brick partially covered with asphalt 0.23, repressed brick tar filler (none above surface of brick) 0.35, repressed brick partially covered with asphalt filler 0.24. The results quoted apply only to the specific surfaces tested and should not be assumed to be typical of all surfaces in the various classes.

One of the main factors influencing safety in highway transportation is the ability of vehicles to stop. Many cities and states have codes specifying stopping distances that must be met. In 1930 a project was undertaken by the Engineering Experiment Station of Ohio State University in conjunction with local police and automobile club officials to determine the minimum stopping distance of automobiles and trucks. These tests showed that if the brakes are properly adjusted, the car will stop well within the allowable limit, and that the stopping distance of any car is dependent upon the type and condition of the road surface as well as the condition and pressure of the tires. After these conclusions had been reached, a second project was begun for the purpose of studying these variations with the hope of obtaining some information that would aid in decreasing highway hazards.

Preliminary tests of the coefficient of friction between tires and road surfaces were made for an undergraduate thesis at the Ohio State

University by R G Kilgore and H N Veley in 1932 A Buick sedan was towed by a truck through a hydraulic dynamometer and the increase in tractive effort was measured when the brake was applied on one rear wheel as the car was towed along at a uniform speed The brake was applied gradually, and when the tire slid the brake was released Due to the limited power and speed of the truck, tests were made only at five and ten miles per hour The results of these tests showed the need of more extensive work on the subject, with equipment that could be operated at much higher speeds

SCOPE OF INVESTIGATION

The coefficient of friction between a tire and a road surface at any instant is dependent upon many variables, such as

(1) Road surface
- a type
- b construction
- c condition (oily, wet, dry)

(2) Speed of operation

(3) Tires
- a. tread design
- b smooth vs good nonskid surface
- c air pressure
- d contact area
- e material

The determination of the effects of all of these variables presents a rather extensive program This paper presents the results to date on a very small part of this outline, namely, the effects on coefficient of friction of different types of wet road surfaces and smooth and nonskid tires Some testing has been done on dry roads, but since wet roads present the real traffic hazard, the comparisons here have been limited to them Tests are in progress on the effects of different tread designs and air pressures, but sufficient data are not available for presentation at this time

The present program has been under way since June, 1932, as a project of the Ohio Engineering Experiment Station, with the assistance of R G. Kilgore, who was granted a two-year Robinson Fellowship on the basis of his undergraduate thesis. The scope of the work has been limited because of insufficient personnel, practically no financial assistance, due to a radical reduction in the University budget, and lack of rain at times when testing could be done It is still hoped that much future work may be carried out with portable and stationary sprinkling systems

APPARATUS

The apparatus consisted of: a trailer, a hydraulic dynamometer, a recording and controlling mechanism 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

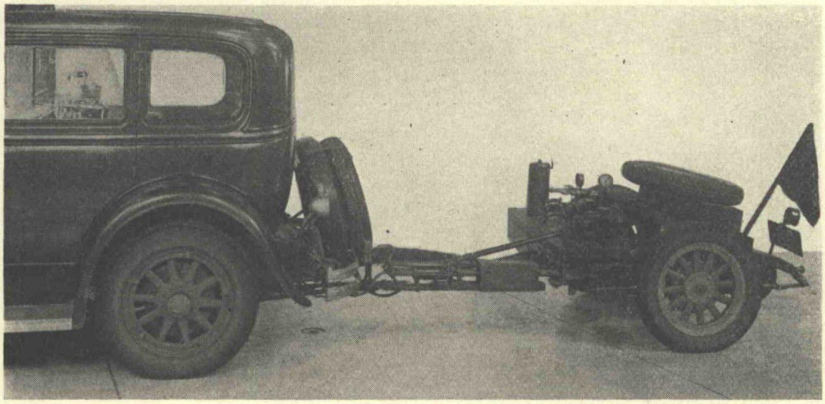


Figure 1. Test Car and Trailer

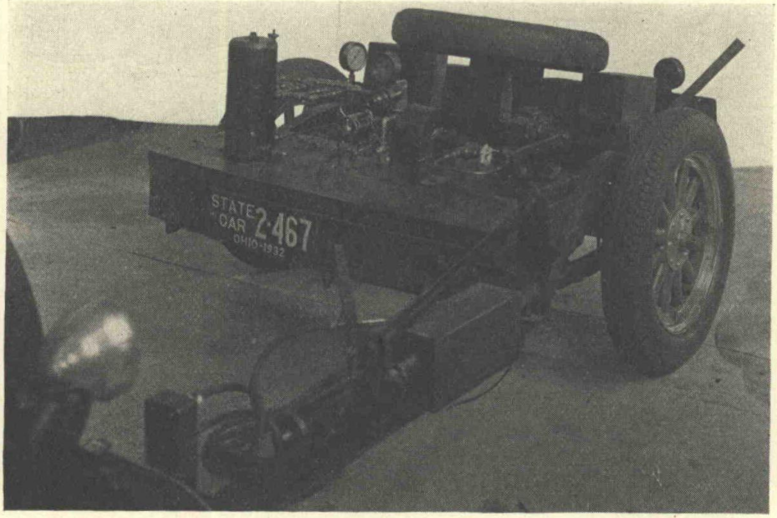


Figure 2. Trailer

line with 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. The original hydraulic brake was used on the wheel and was operated by an air booster

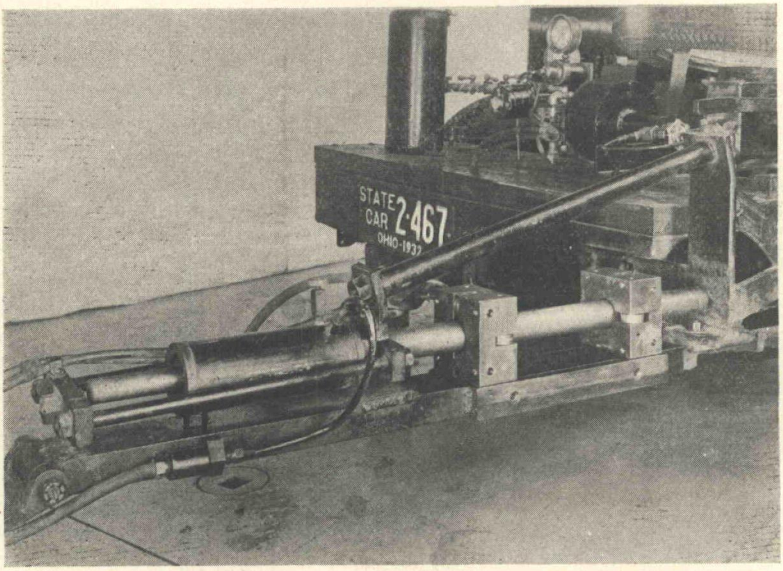


Figure 3. Hydraulic Dynamometer

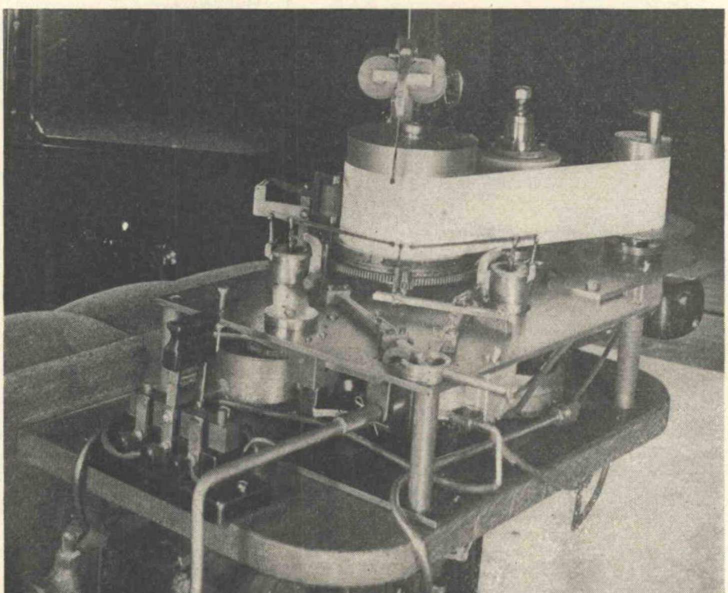


Figure 4. Recording Mechanism

cylinder supplied from a tank at the rear of the trailer. The brake was applied and released by electrical control from the towing car.

Because of the small brake area and the elaborate mechanism required to operate the hydraulic brake, the trailer was recently equipped with a Warner electric brake having direct electric control from the towing car

A six-point cam was mounted on the shaft of the test wheel This cam operated an electric contact every one-sixth revolution of the wheel and produced a record of the initial point of slide.

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 cylinder and piston, and the roller-

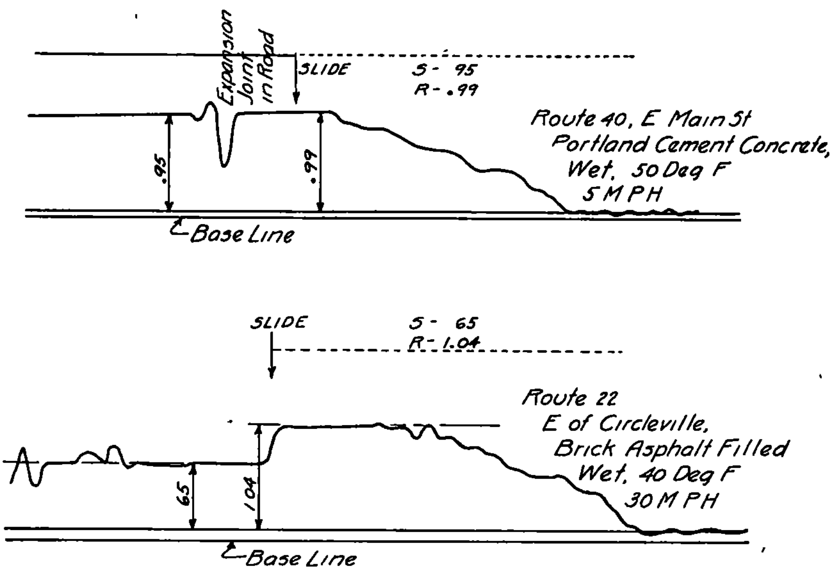


Figure 5. Sample Charts

bearing support The cylinder is bored to a two inch diameter and lapped to fit the piston, no packing or piston rings being used 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 recording and controlling mechanism is mounted in the rear of the towing car The brake is controlled through electric switches The recording mechanism consists of a spring-driven phonograph motor which carries the recording drum Sensitized paper is passed over the drum at approximately two inches per second and is re-rolled after leaving the drum Two records are made on this drum—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 electric breaker points on the test wheel shaft A base line is drawn by a third stylus for use in analyzing the record

METHOD OF TEST

All tests were made during appreciable rainfall, after the dust had been washed from the road surface No tests were made during the initial sprinkling due to the great variation possible, although this condition is known to be the most slippery obtainable The brake mechanism was adjusted to require about two seconds for complete application and the resulting sliding of the tire The air pressure in the test tire was maintained at 32 pounds per square inch unless otherwise specified

The towing car is driven at the desired test speed When speed and road conditions are satisfactory, a signal is given and the operator throws the switch which controls the application of the brake, while the driver regulates the engine throttle to maintain a constant speed As soon as the test tire slides the operator releases the brake With a little practice it is quite easy to maintain a uniform speed during the test At least ten tests are made for each speed in a series, the average of these being taken as the result

The test of any road surface was made with the fundamental idea of obtaining results representing the average condition of the surface The readings were taken while driving continuously over several miles of road when available, no attempt being made to test any small section of surface The road surfaces tested were in most cases practically new and no variation in results could be noted when testing in or out of the traffic lanes

CALCULATION OF RESULTS

The initial tractive effort is indicated on the sample charts shown, (Figure 5) as the distance above the base line at the right end of the record This line is extended parallel to the base line The distance above this line to the maximum height of the curve before sliding is a relative measure of the rolling coefficient of friction while the average height of a two inch section of the curve beyond the point of sliding is taken as the measure of the sliding coefficient This average height is determined from the planimetered area The accompanying calibration curve shows the oil pressure and tractive effort for the movement of the recording stylus When the net height of the curve is obtained from the test chart, the net tractive effort can then be found from the calibration curve The coefficient of friction $= \dfrac{T E}{W - 0\ 2\ T E.}$ where W is the dead load on the test wheel In this case it was 801 pounds A net height on the chart of one inch represented a tractive effort of 504 pounds and $\dfrac{504}{801 - 0\ 2 \times 504} = 72$

DESCRIPTIONS OF ROADS TESTED

T-50-D-16 East Broad Street in Bexley This is a petroleum asphalt road surface on a concrete base It was completed in October, 1933. The surface of $1\frac{1}{4}$ inches sheet asphalt (Type D, Ohio State Highway Specifications 1933) was built of materials conforming to the following specifications

Sand

Passing a No 4 sieve	100%
Passing a No 4 sieve, retained on a No 8 sieve	0–5%
Passing a No 8 sieve, retained on a No 40 sieve	12–40%
Passing a No 40 sieve, retained on a No 80 sieve	25–60%
Passing a No 80 sieve, retained on a No 200 sieve	25–45%
Passing a No 200 sieve	0–5%

Bituminous Material

Specific gravity 25°C / 25°C, not less than	1 01
Flash point, not less than	200°C
Penetration at 25°C, 100g —5 sec	50 to 60
Ductility at 25°C, not less than	50 cm
Loss at 163°C 50 g —5 hours, not over	1%
Penetration of residue at 25°C, not less than % of original	60%
Total bitumen (sol in CS_2) not less than	99 5%

No cement was scattered on the machine-finished surface

T-5-40 Hebron to Jacksontown on U S 40 This portland cement concrete road was resurfaced with medium texture hot-mixed, hot-laid bituminous concrete (Ohio State Highway Specification T-5, Type B 1932) The composition of the mixture by weight was as follows

Passing screen or sieve	Retained on screen or sieve	Per cent		
		Minimum	Ideal	Maximum
$\frac{3}{4}$ Inch	$\frac{1}{2}$ Inch	0	0	5
$\frac{1}{2}$ Inch	$\frac{1}{4}$ Inch	41	48	55
$\frac{1}{4}$ Inch	No 10	0	5	10
No 10	No 20	8	12	15
No 20	No 50	12	16	20
No 50	No 100	2	9	13
No 100		0	3	5
Total stone content retained on No 10 sieve				45 to 55%
Bitumen		6 5		8 5

The road was completed in November, 1932

T-6-22 U S Route 22, east of Circleville This is a cold-mixed, cold-laid bituminous concrete road which was completed in the autumn of 1932 (Ohio State Highway Specification T-6 Type B, 1932) The composition of the surface course was as follows:

Stone Size	No 6
Coarse Aggregate	67 0 to 89 5 per cent
Fine Aggregate	5 0 to 25 0 per cent
Liquefier	0 0 to 1 5 per cent
Bitumen	5 0 to 8 0 per cent

BA-40. East Main Street in Columbus and east of Bexley This surface consists of a three-inch, wire-cut vertical-fiber lug brick filled and covered with type F-1 asphalt (Ohio State Highway Specification Type F-1) The road was completed in November, 1931 Sand was placed on the asphalt during the summer of 1932

BA-23 North High Street, Columbus, between Arcadia and Oakland Park Avenues on U. S Route 23 This street was constructed of three-inch repressed brick filled and approximately 25 per cent covered with asphalt. The paving was completed about 1926

BA-22 East city limits of Circleville on U S Route 22 Three-inch vertical-fiber wire-cut lug bricks with asphalt filler (Ohio State Highway Specification Type F-1) were used for the surface The surface of the bricks was white-washed before pouring the filler so as to permit removal of the excess

BO-23 Between Worthington and Columbus on U S Route 23 This road was constructed about 1920 and consisted of a repressed brick with a grout filler The filler has broken out so that it is about one-half inch below the surface of the bricks. The edges of brick are badly chipped

BO-OP. Oakland Park Avenue, Columbus, between High Street and Indianola Avenue. On this street repressed brick with tar filler gives a surface similar to BO-23 due to the fact that the filler has gone down However, the edges of the brick are not broken so as to give a rounded surface on each brick

C-40. U. S Route 40, east from the Columbus city limits This 50-foot portland cement concrete road between Columbus and Reynoldsburg was completed in December, 1930 Crushed gravel aggregate was used in a 1:5.5 mix The surface was machine finished.

DISCUSSION OF RESULTS

A typical range in results is shown in Figure 6, for tests of rolling and sliding friction of a nonskid tread design tire on a bituminous concrete surface The shaded areas show the limits of variation The occasional tests that depart widely from such range limits can generally be traced to a radical change in road surface or other test conditions

Several sets of tests have been made on some of the test roads to see if the results could be duplicated and also to observe the variation of the coefficient of friction with age The only changes noted in the tests so far are shown in Figures 11 and 12 between curves BA-22-32

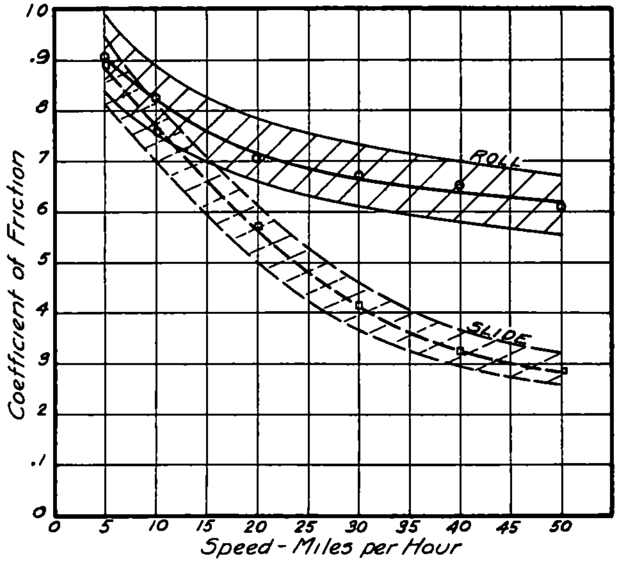


Figure 6. Friction Range of Tire HT (Non-skid) on Portland Cement Concrete Resurfaced with Bituminous Concrete, Type T5, February 7, 1933.

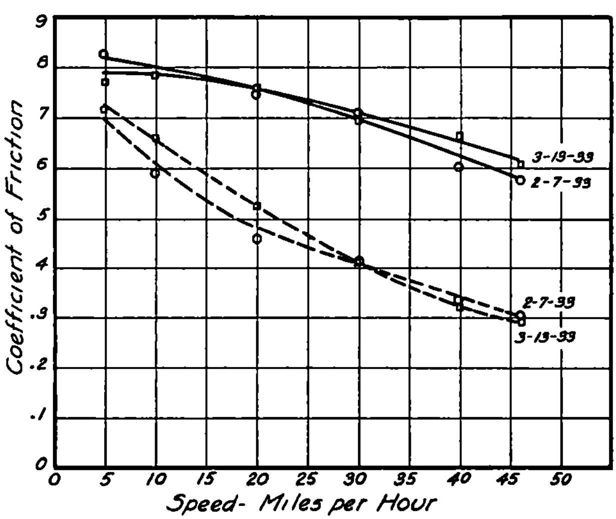


Figure 7. Comparison of Tests on Portland Cement Concrete (C 40), Non-skid Tire HT, February 7 and March 13, 1933.

and BA-22-33 which show the effects of summer bleeding of asphalt. Figure 7 shows the agreement obtained on two series of tests made on C-40 about one month apart.

The difference in coefficient of friction between smooth and nonskid tires was investigated A few tests have been made using two

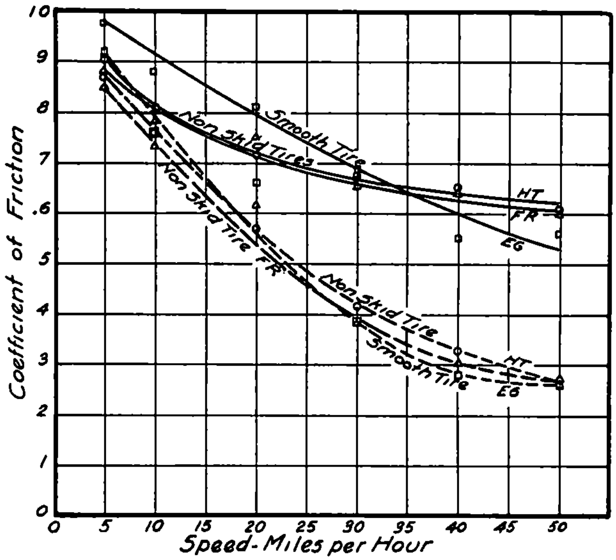


Figure 8 Rolling and Sliding Friction Variation on Bituminous Concrete Surface T-5-40. March 3, 1933. Solid Lines, Rolling, Dash Lines, Sliding

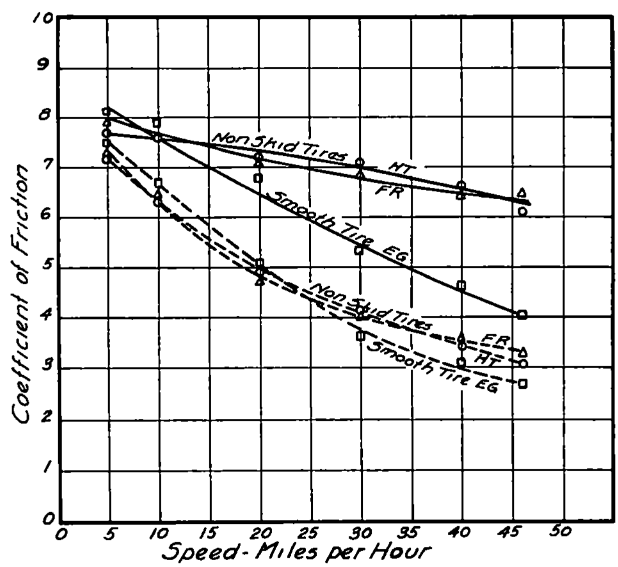


Figure 9. Rolling and Sliding Friction Variation on Portland Cement Concrete Surface, C-40. March 13, 1933. Solid Lines, Rolling, Dash Lines, Sliding.

designs of nonskid tread and a smooth tire The nonskid tread design tires (HT and FR in the figures) have a design consisting of three ribs.

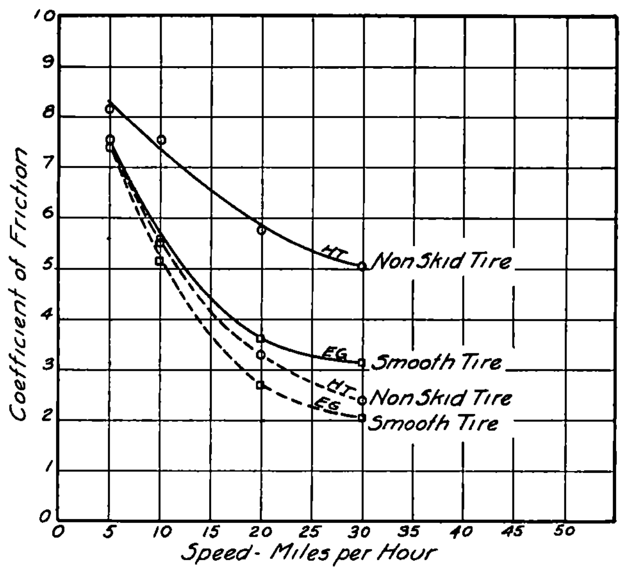


Figure 10 Rolling and Sliding Friction Variation on a Repressed Brick Surface, Filled and Approximately 25 Per Cent Covered with Asphalt. March 13, 1933. Solid Lines, Rolling, Dash Lines, Sliding

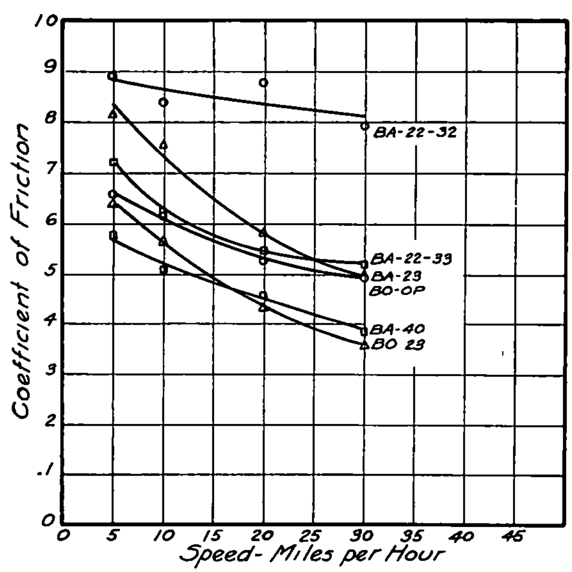


Figure 11. Rolling Friction on Brick Pavements With Non-Skid Tread Tire HT, November 25, 1933.

BA-22. Vertical Fiber, Wire Cut Lug Brick. Excess Asphalt Filler Removed.

BA-23. Repressed Brick, Approximately 25 Per Cent Covered with Asphalt Filler.

BA-40. Vertical Fiber, Wire Cut Lug Brick, Covered with Asphalt Filler, Sanded.

BO-OP. Repressed Brick, Tar Filler.

BO-23. Repressed Brick, Grout Filled, Grout Below Surface.

The center rib has nonskid patterns on each side while the outer ribs have nonskid patterns on the outer edges only. On tire HT the nonskid edges are normal and parallel to the direction of travel, while on the FR tire the edges are at a 45 degree angle Tire EG is worn to an abnormally smooth tread The results are shown in Figures 8, 9, and 10.

The rolling coefficient of friction does not vary radically on the T-5-40 bituminous concrete road, but on the C-40 portland cement concrete and the BA-23 asphalt-filled brick roads there is a decided decrease

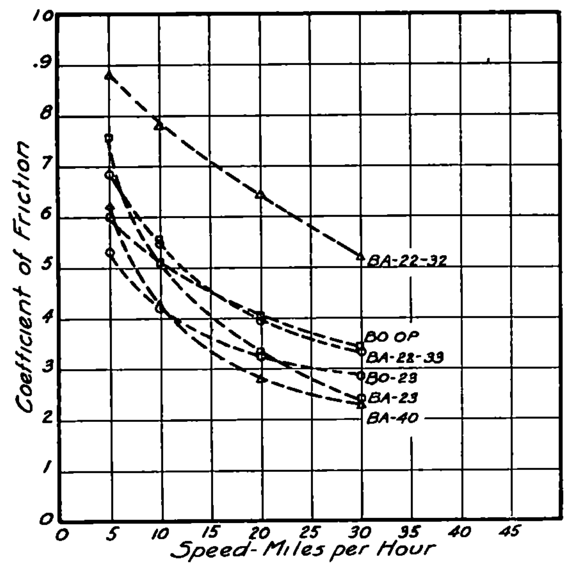


Figure 12. Sliding Friction on Brick Pavements, with Non-Skid Tread Tire HT, November 25, 1933.

BA-22. Vertical Fiber, Wire Cut Lug Brick. Excess Asphalt Filler Removed

BA-23. Repressed Brick, Approximately 25 Per Cent Covered with Asphalt Filler.

BA-40. Vertical Fiber, Wire Cut Lug Brick, Covered with Asphalt Filler, Sanded.

BO-OP Repressed Brick, Tar Filler.

BO-23. Repressed Brick, Grout Filled, Grout Below Surface.

when using the smooth-tread tire "EG" This might be due to the more open structure of the road surface The sliding coefficient of friction is more nearly the same for all tires, the variation at high speeds being in favor of the nonskid tires on all three roads

Figures 11 and 12 show the rolling and sliding coefficients of friction found on several brick road surfaces The variation between a vertical-fiber brick road, BA-22-32, free of asphalt filler, and a repressed brick road, BO-OP, here termed "open," as no filler was visible, is shown by comparing the curves for these two roads The repressed brick has rolling and sliding coefficients of friction at thirty miles per hour which are about 61 and 65 per cent respectively of those obtained for the vertical-fiber brick

During the summer of 1933 the asphalt filler bled from the joints on road BA-22 and covered in some places as much as 50 per cent of the road surface. The results of the tests on the original road are shown in curves BA-22-32 while the results of a series of tests on the same road made in October, 1933, are shown in curves BA-22-33 During the summer the rolling and sliding coefficients of friction at thirty miles

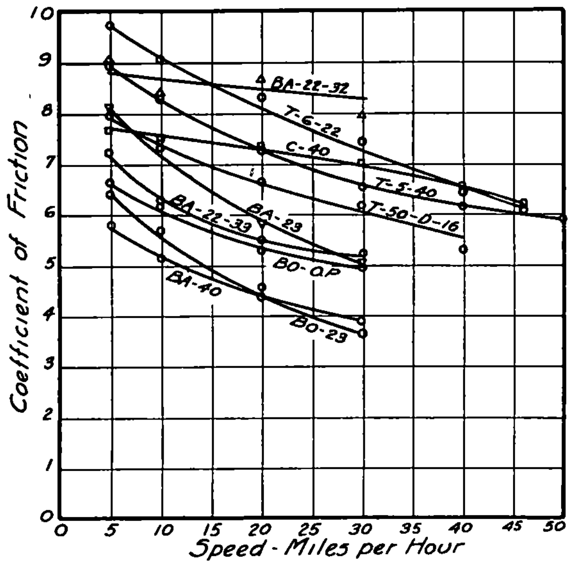


Figure 13. Rolling Friction Variation, with Non-Skid Tread Tire HT, November 25, 1933

BA-22 Vertical Fiber, Wire Cut Lug Brick. Excess Asphalt Filler Removed

BA-23. Repressed Brick Approximately 25 Per Cent Covered with Asphalt Filler.

BA-40. Vertical Fiber, Wire Cut Lug Brick, Covered with Asphalt Filler, Sanded.

BO-OP. Repressed Brick, Tar Filler.

BO-23. Repressed Brick, Grout Filler, Grout Below Surface.

C-40. Portland Cement Concrete.

T-5-40 Bituminous Concrete.

T-6-22. Cold Mix, Cold Laid Bituminous Concrete.

T-50-D-16. Sheet Asphalt

per hour decreased to about 64 and 62 per cent respectively, of the original values

Another vertical-fiber brick road, BA-40, was flooded with asphalt filler and the filler not removed, but heated and sprinkled with sand which was then rolled into the asphalt

The nonskid quality of the vertical-fiber brick was completely lost and the road can be seen to have a much smaller coefficient of friction, both rolling and sliding, than either BA-22-32 or BA-22-33.

The curves for roads BO-OP and BO-23 show comparative results on a good open repressed brick road and one that was 13 years old and very badly chipped The good road, BO-OP, has much larger coefficients at all speeds

A comparison of the curves for roads BA-23 and BO-OP will show the variations of the coefficients of friction between repressed brick road surfaces (a) when using asphalt filler with the surface fairly free of asphalt—not over 25 per cent covered (BA-23) and (b) when the

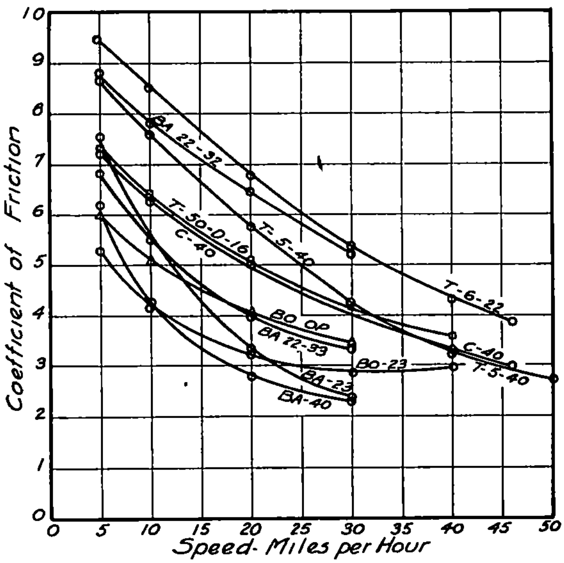


Figure 14 Sliding Friction Variation with Non-Skid Tread Tire H T, March 13, 1933

BA-22. Vertical Fiber, Wire Cut Lug Brick Excess Asphalt Filler Removed.

BA-23 Repressed Brick Approximately 25 Per Cent Covered with Asphalt Filler.

BA-40. Vertical Fiber, Wire Cut Lug Brick, Covered with Asphalt Filler, Sanded.

BO-OP Repressed Brick, Tar Filler.

BO-23. Repressed Brick, Grout Filler, Grout Below Surface.

C-40. Portland Cement Concrete.

T-5-40. Bituminous Concrete.

T-6-22. Cold Mix, Cold Laid Bituminous Concrete

T-50-D-16. Sheet Asphalt.

joints are open (BO-OP) The values for the rolling coefficient are better at low speeds for BA-23 but at 30 miles per hour both roads have the same value, while the sliding coefficient is much better for BO-OP above 15 miles per hour

The three bituminous-concrete road surfaces tested show a variation of rolling coefficient of friction in Figure 13 and this difference is in favor of the open types of surface in the order of T-6-22, T-5-40, and T-50-D-16 The surfaces of the T-5-40 and T-6-22 roads have about the same degree of roughness and open structure Portland-cement concrete, C-40, curve shows much less slope, at low speeds having the same value as T-50-D-16 while at 40 miles per hour and over it is at least equal to T-6-22

On Figure 14 the sliding coefficient of friction curves for T-5-40 and T-6-22 roads are seen to be parallel but favorable to T-6-22 T-50-D-16 has a lower sliding coefficient at low speeds than either of these but as the speed increases the slope of the curve is much less This is also true of the portland-cement concrete road, C-40, the curve being practically identical with the T-50-D-16 curve, both curves being at least equal to the T-5-40 beyond 40 miles per hour This might be due to the greater uniformity of the T-50-D-16 and C-40 surfaces

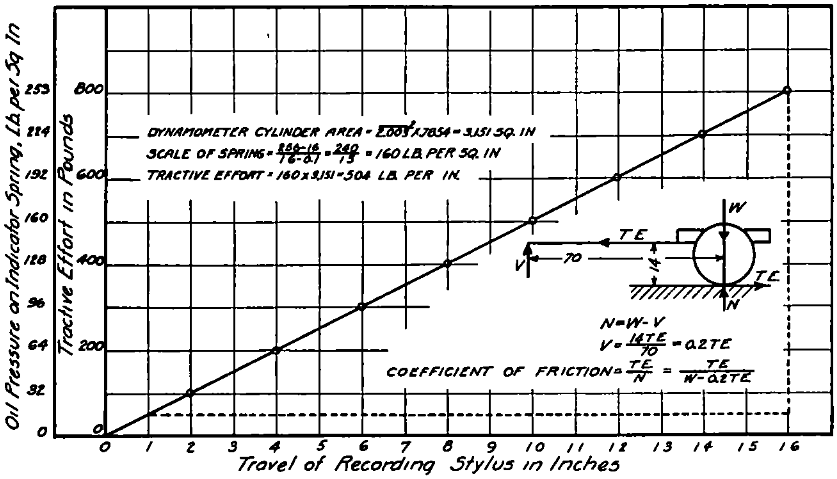


Figure 15. Calibration of Dynamometer Pressure Recording Mechanism October 26, 1932.

A comparison of portland-cement concrete, C-40, and sheet asphalt, T-50-D-16 can be seen on Figures 13 and 14 The rolling coefficients of friction are about the same at low speeds, but at higher speeds the C-40 maintains higher values The sliding coefficients are very interesting as they are practically identical up to 30 miles per hour when there is a slight advantage for the T-50-D-16

The rolling and sliding coefficient of friction curves for new vertical-fiber brick road surface BA-22-32 are well above those just mentioned for C-40 and T-50-D-16 However, after the bleeding of the asphalt, the curves BA-22-33 are seen to be some distance below the portland-cement concrete and sheet asphalt curves

From the standpoint of safety there are two trends shown by these tests that are worthy of consideration (1) The coefficients of friction on all road surfaces decrease with increase of speed (2) The difference between the rolling and sliding coefficients increases with increase of speed Considering these points, the driving hazards are much more serious on wet roads than on dry roads as the speed is increased, not only because of the decrease in the rolling as well as sliding coefficients of friction, but also on account of the increasing difference between the values of rolling and sliding coefficients of friction This latter variation is exceedingly serious when brakes are applied suddenly as any type of brake may operate unequally at times on the four wheels and when one wheel slides there is a radical decrease in braking force on this wheel, which may easily cause the driver to lose control of the car

DISCUSSION

ON

SKIDDING TESTS AND COEFFICIENTS OF FRICTION

MR E O RHODES, *Koppers Products Company.* I would like to call attention to certain points that need to be stressed in work of this kind, so that they may be considered in the continuation of these experiments

It is extremely important that the history be given in detail for each road surface tested—the type of the road, type of binder, specifications for the binder, method of construction or treatment, age, and surface condition at the time of test Also the records as to treatment subsequent to construction should be included, for we know there is a possibility that some of these road surfaces may have had several kinds of materials placed upon them

Also, I think it is important that this work be continued by making a great number of tests on each type of surface

The matter of testing in center of road as compared to testing in the traffic lanes should be considered Actually we are principally interested in the condition of the surface on which we are doing most of our riding If we go to the center of the road, are we testing the surface which is carrying most of the traffic?

Whether the pavements are wetted by rain or artifically, the prevailing conditions, such as time, the amount of rainfall, the weather conditions prior to time of tests, and temperature prior to time of tests, are important factors and need to be considered

That some of the coefficients which have been presented do not agree very satisfactorily with coefficients that have been determined by other experimenters in other countries is probably not so much due to the differences in apparatus as it is due to the differences in surfaces of the same type

MR E M FLEMING, *Portland Cement Association* Coefficients of surface friction are only valuable for practical application in so far as they measure the relative safety of the various surfaces The question naturally arises as to which of the three coefficients mentioned is the critical one.

In the report the coefficient of straight skidding is given most weight. This, together with the sideways skid coefficient, is a measure of the ease and rapidity of skidding *after* the action has already started. Very little is said about the coefficient of *impending* skid, which measures the ability of the surface to set up resistance to any skidding motion.

It seems to me that the coefficient of impending skid is more nearly a measure of surface safety than either of the two others.

In one chart the impending skid coefficient on concrete is about 0.80 at 40 miles per hour. Under the same conditions the coefficient for sideways skid is 0.60 and for straight skid about 0.40. While all three coefficients at low speeds (five miles per hour) are approximately equal, yet the impending coefficient remained practically constant with increasing speed while the others decreased rapidly.

Some discussion of the relative importance of the three coefficients in so far as they measure driving safety would be desirable. It would be interesting to know for other types of surfaces the relation of the impending coefficient to the side and straight coefficients and also to compare the values of this coefficient at various speeds on those surfaces with the ones on concrete.

The Ohio skid tests show clearly that the maximum rolling skid coefficients, or perhaps, more properly, impending skid coefficients, are much higher on all types than the straight skid coefficients. As previously mentioned, the impending skid coefficients are the true criterion of skidding accidents since the car must first start to skid before the sliding skid coefficients govern.

The curve comparing rolling friction or impending skid coefficients on various types of pavements shows that concrete has the highest values for speeds of 40 miles an hour and greater-speeds at which skidding accidents may be dangerous. The same trend of increase in sliding friction on concrete above other types is also indicated at the higher speeds, although the tests do not cover this point as fully as tests on rolling friction.

In comparing the Ohio and Iowa skid tests, it will be noted by the Iowa tests that the close-textured bituminous types have higher values than the open-textured bituminous types, yet by the Ohio tests, the opposite condition was found. Since only a specific project was tested in each case, it indicates that additional work is needed to establish the comparative skid coefficients on the open and close-textured bituminous types as well as with other types.

MR. BERNARD E. GRAY, *The Asphalt Institute*: In making further studies of the skid resisting qualities of road surfaces, it is suggested that account be taken of the character of aggregate composing the pavement It is recognized that a considerable variation exists in this regard, particularly in the case of the cover coat aggregates used in low cost types The endeavor of the highway builder should be to have traffic carried on the aggregate rather than upon a film of bituminous material, and certain aggregates under traffic tend to become polished and slippery whereas certain other aggregates, because of a rough surface texture, have a high resistance to skidding

Mr George E Martin, *The Barrett Company* · While these tests give results for the individual sections where the work was done, the data are not sufficient to justify conclusions as to the relative skid-resistance of various pavement types

Many of the variables which influence the results are not included in this report Some of these are the detailed method of construction of the surface, its age, previous maintenance history, and kind and amount of traffic

The report does not give the results of individual tests but only a summary for a particular pavement type It is impossible to tell, therefore, how much variation there may have been between individual results

Motor vehicles skid on the driving wheels and there is no certainty that the results for pulled wheels would be an accurate measure of those for driving wheels

The value of the coefficient of impending skidding, which is undoubtedly of most importance to the motorist, is reported for only a very few experiments

The method of producing a wet pavement gives results which are only partially comparable to natural wet weather conditions

This report is a good progress report on one method of approaching the problem but cannot be considered as giving final results relative to the comparative skid resistance of various roadway surfaces

Mr G F Schlesinger, *National Paving Brick Association* The report of Professors Stinson and Roberts shows that on a pavement constructed with a vertical fiber lug type of brick and from which the filler had been removed at the time of construction the paving surface had a high coefficient of friction—one of the highest of the different types represented by the curves shown This skid-proof quality was reduced when the same pavement was tested a year later with the automobile traveling in the main line of traffic, although it still had a fairly high coefficient of friction This reduction was due to the asphalt filler having risen in the joints and spread over a portion of the brick surface It is my opinion, however, that, if this same pavement is tested from year to year, the coefficient will gradually be restored to its original value because of the disappearance of asphalt from the surface under the action of traffic

Professor Moyer tested some brick pavements in the city of Des Moines, Iowa, and, according to his results, the pavement with about 25 per cent of asphalt on the surface had more friction than the clean brick pavement This is inconsistent with the tests of Professors Stinson and Roberts, and, in my opinion, is not according to practical experience I have investigated several brick pavements concerning which there had been complaints regarding slipperiness In every instance, these pavements were covered with asphalt in which the mineral cover material had not been properly incorporated The National Paving Brick Association recommends the surface removal method of filler application and the vertical fiber lug type of brick believing that these requirements will produce the most non-skid qualities for the brick type of pavement

Professor Moyer, *Author's Closure, by Letter* In the light of the present knowledge of the subject, it appears that the facts and principles established in this paper are fundamentally sound and should form the basis for future investigations The suggestions made in the discussion should prove helpful in carrying out such a program The differences in the coefficient of friction due to variation in the types of asphalts, tars, aggregates, mineral fillers and other road surfacing materials now in use can for the most part only be determined accurately by actual test Since it would require too extensive a program to determine the effect of all these variables, it was thought desirable for the present to restrict the program to work from which it would be possible to rationalize the results obtained in the tests and to formulate a theory by which the effects of skidding might be predicted for any given set of conditions

In the cases where tests were made in the center of the road, it was possible at the same time to test in the center traffic lanes. The purpose for testing in the center of the road was largely to eliminate the effects of crown

The writer takes exception to the statement that the results of tests reported in this paper do not agree with results obtained in other countries Probably the most outstanding piece of work along this line was that conducted by the National Physical Laboratory of England during 1930 In a paper by Bradley and Allen[1] data are reported for about ten types of surfaces These data are in substantial agreement with the data reported in this paper in the cases where the types of test and surfaces conditions are similar

The side skid and straight skid coefficients are most likely to be critical coefficients for the reasons previously stated in the paper The

[1] "Factors Affecting the Behavior of Rubber-tired Wheels on Road Surfaces" by J Bradley and R F Allen, 1930–31, Proceedings of the Institution of Automobile Engineers

coefficients for impending skidding can hardly be considered critical, since the brakes on comparatively few cars, are adjusted uniformly enough to make available this high coefficient at each of the four wheels It is also quite likely that few drivers can apply brakes so skillfully that they can bring the four wheels to the point of impending skidding It should be observed, however, that the side skid and skidding impending coefficients are generally about the same for regular balloon tires

The coefficients for the brick and concrete surfaces were approximately the same with the coefficients for brick slightly lower than those for concrete Brick and concrete have certain characteristics in common which in part accounts for the similarity in their skidding actions An important common characteristic is that these surfaces are hard and dense The cementing or bonding materials are exceptionally hard and for this reason a slight polishing effect can be observed on these surfaces due to the abrasive action of the tires Althougn these surfaces may have an exceptionally high coefficient when new, the tests on surfaces which have been subjected to traffic indicate that this rough-textured condition will not exist for a great length of time after the road is open to traffic Nevertheless, the surface texture on these surfaces should never be so smooth as to cause the coefficients on the wet surfaces to be dangerously low It is quite likely, that the type of bituminous filler and the extent to which fine aggregate and mineral filler are present in the bituminous filler of brick surfaces, are responsible for the large variable effect in the skidding characteristics of brick surfaces

INTANGIBLE ECONOMICS OF HIGHWAY TRANSPORTATION

By C A Hogentogler, E A Willis, and J A Kelley

United States Bureau of Public Roads

SYNOPSIS

This discussion suggests changes in the physical structure of routes and terminal facilities for utilizing, in urban transportation, the comfort, speed, safety, and convenience made possible by progress in the design of motor vehicles and the road surfaces

Special attention is called to the importance of terminal facilities Improvements in car and highway design have done much to increase comfort, speed, and safety of travel Terminal progress, in contrast, has principally concerned itself with legal restrictions as a result of which the demand for urban parking space greatly exceeds, by 5 or 10 to 1 in some instances, the effective supply This robs the car owner of much of the benefit for which he has demonstrated not only his ability, but his eagerness to pay

In considering the economics of terminal facilities, the important question concerns the expenditure justified by sound economics to increase the speed, comfort, and safety of the entire traveling public by removing parked cars from the highly expensive street surfaces to off curb parking areas with less expensive surfacing Parking space is not merely a personal convenience for car operators, but an integral part of economic motor transport

Intangible, according to Webster, means "not directly appreciated by the mind" Economics, according to the same authority, pertains among other things to "means or methods of living well" Hence, our discussion concerns the imperceptible personal and social benefits furnished by private motor transport

Frugality has no place in the picture Whether travel via motor costs more or less than that furnished by rail, boat or street car has little significance here

Of more importance are the social improvements for which the traveling public have demonstrated their eagerness regardless of cost These include the transformation of the urbanite to the suburbanite, the remodeling of rural educational systems, the making of the urban commercial facilities available to the farm, the rural recreational facilities to the city, the mountains and the seashore to both; and withal the exhilaration and rehabilitation which contribute not only to our pleasure but our physical and mental health as well.

Financial returns can no more evaluate such benefits than those furnished by the motion picture, the theater, the baseball game or the pleasure trip abroad Their costs are not justified by frugality but instead by adequate return in the way of better living, intellectual improvement, and recreation productive of increased working capacity Undoubtedly, such intangible benefits account largely for the huge contributions of passenger car transport to the nation's commerce

Facts and figures of the automotive industry, for instance, disclose that about 20 million passenger cars were registered in the United States in 1932 At an assumed low purchase price of $600 each they must have put about twelve billion dollars in circulation during the past five or six years

Also, if the cost of operation and replacement can be assumed at the conservative figure of five cents per mile and the annual travel at 7,500 miles, seven billion dollars are poured into the business channels of the nation annually H J Struth notes in American Motorist, December, 1929, that the total gas, gas tax and oil bills alone amount to $6,000 per minute

A problem in the field of structural design is thus introduced It suggests the necessity of changing the physical structure of route and terminal in order to furnish the motorist with the comfort, speed, safety and convenience for which he has demonstrated not only his ability but his eagerness to pay

In this connection let us consider the case of one of the 7,000,000 passenger car owners residing in 124 cities of population exceeding 50,000 each and with a total of 38,000,000 Furthermore, let our driver be one of the 143,000 registrants in the City of Washington

While much of his mileage costs less than that charged by commercial carriers, yet some of it undoubtedly costs more His willingness to pay the additional cost of the latter may be explained as follows

To ride in the commercial carrier he *must* be willing to board it at some point on its route, at its scheduled time, must travel only on the streets it traverses, with whatever in the way of standing or seating facilities it offers, and finally must be satisfied to get off at some point along its route closest to his destination

His own car, in contrast, leaves from his residence, at his convenience, travels at such speed and along such route as he desires, and lands him at his destination with the utmost despatch and comfort

The difference in accommodations is both physical and mental Physical, as to comfort, convenience, etc , and mental as to regulations and restrictions Four or five "musts" which control his travel in commercial carriers are eliminated by the private transport

The known aversion of the American people to any but the most essential regulations suggests that desire to escape these "musts" or mental restrictions accounts largely for the huge private car ownership, that freedom from the irksome restrictions, not to mention possible physical discomforts of commercial carriers, is largely responsible for the assumed 140 billion miles of annual passenger car transport

The essential features desired in this type of motor transport are convenient origin, comfortable and safe transit and satisfactory terminal.

Figure 1A

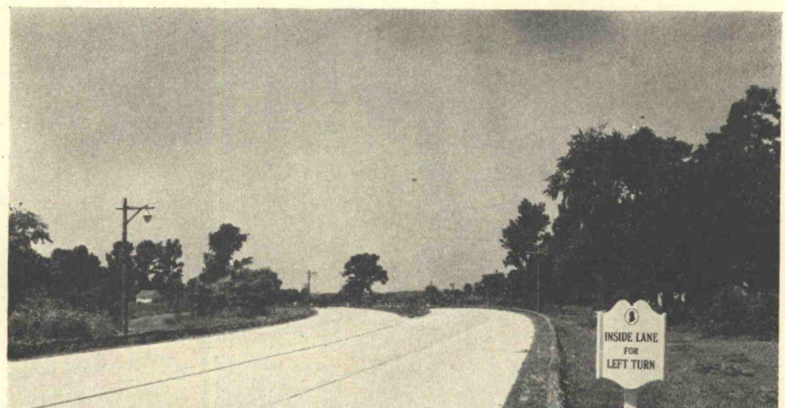


Figure 1B

Resident emigration to suburban districts has done much to provide origin facilities; ingenuity of motor car design combined with increased road widths, better alignment, easier curves with greater sight distances and modern traffic control methods and mechanisms have done much to expedite safe transit.

The destination or parking space, in contrast, seems to have been left out of consideration, causing progress in terminal facilities to consist in the main of legal restrictions. Even current technical literature

Figure 2A

Figure 2B

reflects the neglect of terminal facilities. Thus, for instance, in a bibliography consisting of 802 articles on traffic engineering but 47 concerned parking and, of these, 41 had to do with regulations, leaving but six to deal with actual terminal structures or parking space.

Figures 1A and 1B show how engineering skill transformed the dusty, muddy part-season road of 20 years ago into the ultra modern high speed superhighway of today.

Figures 2A and 2B show how the bridge builder contributed his share to the improvement in transportation facilities.

Figure 3A

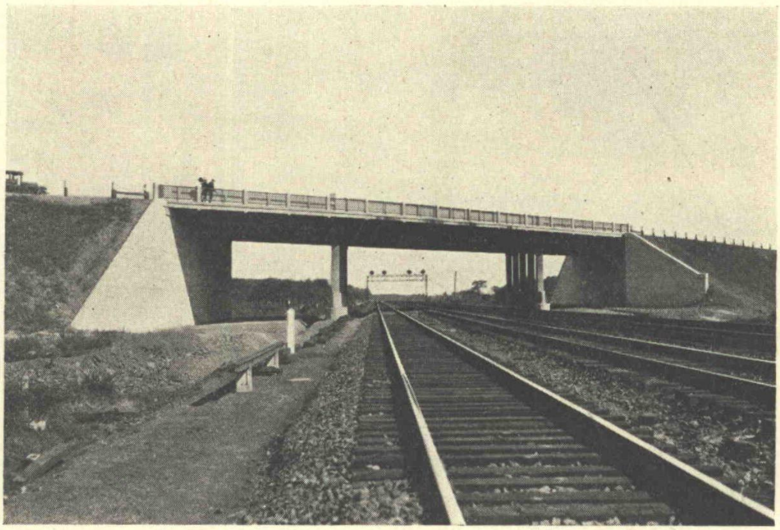


Figure 3B

Figures 3A and 3B illustrate how change in physical structure as substitute for regulatory signs, signal, etc. is solving the grade crossing problem.

Figures 4A and 4B illustrate how the genius of the automotive engineer transformed the stop-and-go, 15-mile-per-hour horseless carriage of a quarter century ago into the dependable, high speed, luxurious modern palace of comfort.

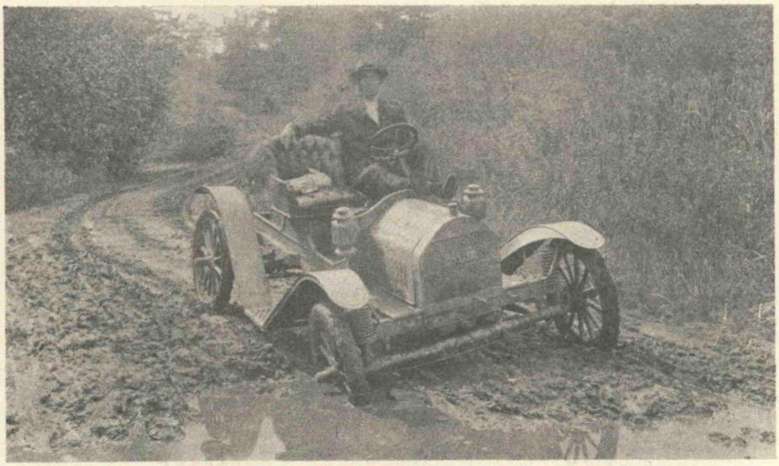


Figure 4A

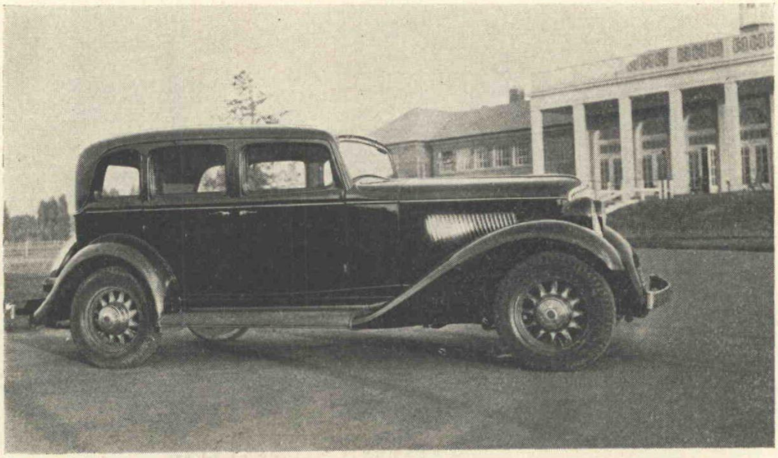


Figure 4B

Figures 5A and 5B show that while increased facilities are made for the traveling car, the improvement does not increase the parking space, the sacred curbs remaining inviolate. Here the parkway in the center was eliminated and the width between curbs (Figure 5A) was increased to furnish better facilities for moving vehicles. As seen in Figure 5B, however, the improvement furnished no increase in the facilities for parking cars.

Figures 6, 7 and 8 illustrate the effect of the absence of terminal facilities. They were obtained by recording the times by means of a

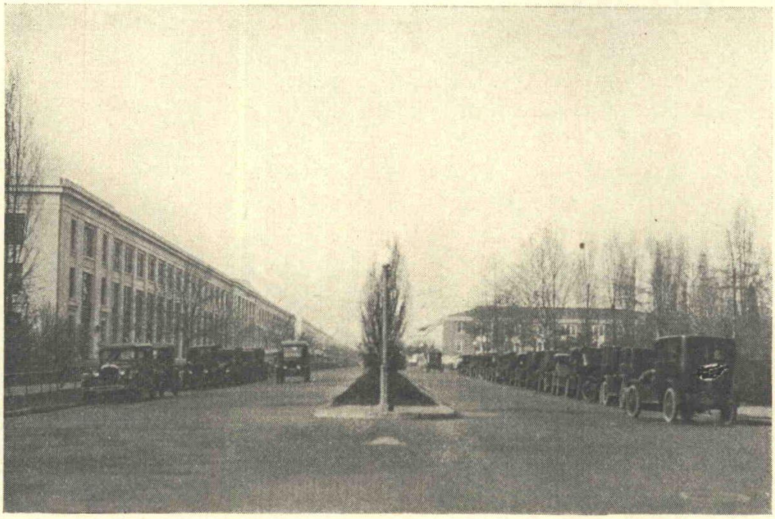


Figure 5A

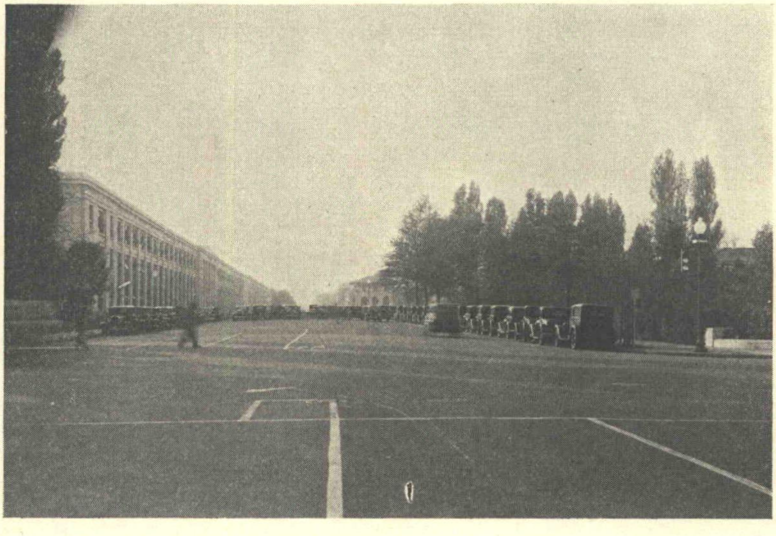


Figure 5B

stop watch corresponding to odometer readings on different routes during the morning rush hour.

Figure 6 shows the run down Sixteenth Street in Washington. The distance traveled is plotted against time. The distance from origin to destination is six miles. The upper curve shows that at 5 a.m., in the absence of travel, almost the maximum legal speed is maintained, thus allowing the trip to be made in 14 minutes

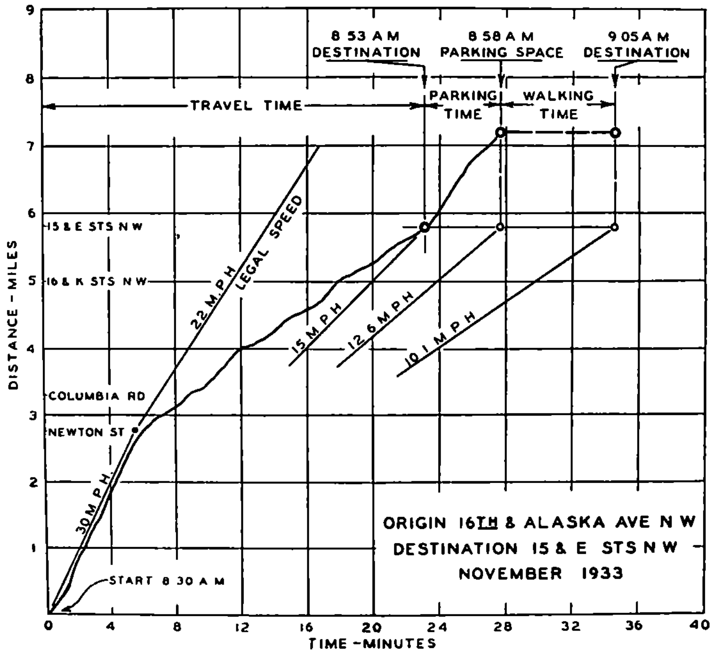


Figure 6. Effect of Lack of Parking Facilities upon Trip Time

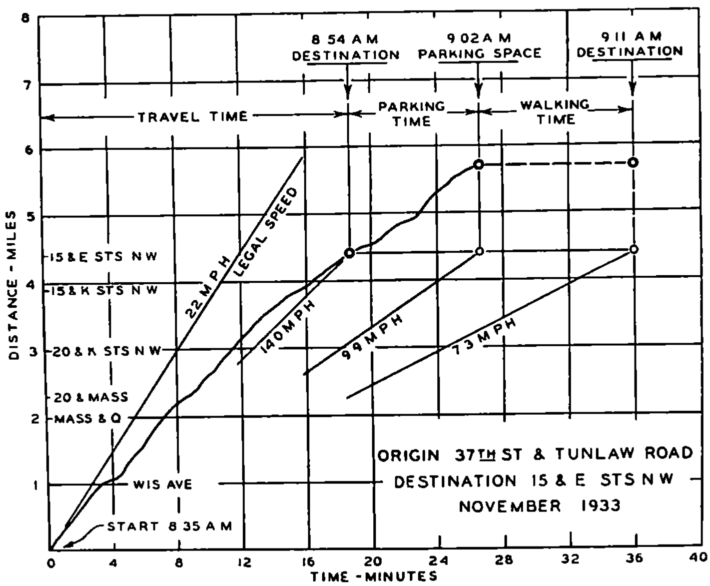


Figure 7. Effect of Lack of Parking Facilities upon Trip Time

During the rush hour, however, the average speed from origin to destination is reduced to 15 miles per hour, requiring 23 minutes At the end of five additional minutes spent in finding a parking space it drops still further to 12 6 miles per hour, and an additional seven minutes spent in walking back to the destination brings the final average to 10 1 miles per hour

Figure 7 shows a similar run from 37th Street and Tunlaw Road to the Commerce Building Here the speed to destination, 14 miles per hour, is reduced to 9 9 after the car is parked and finally to 7 3 miles per hour at the conclusion of the walk back to the office

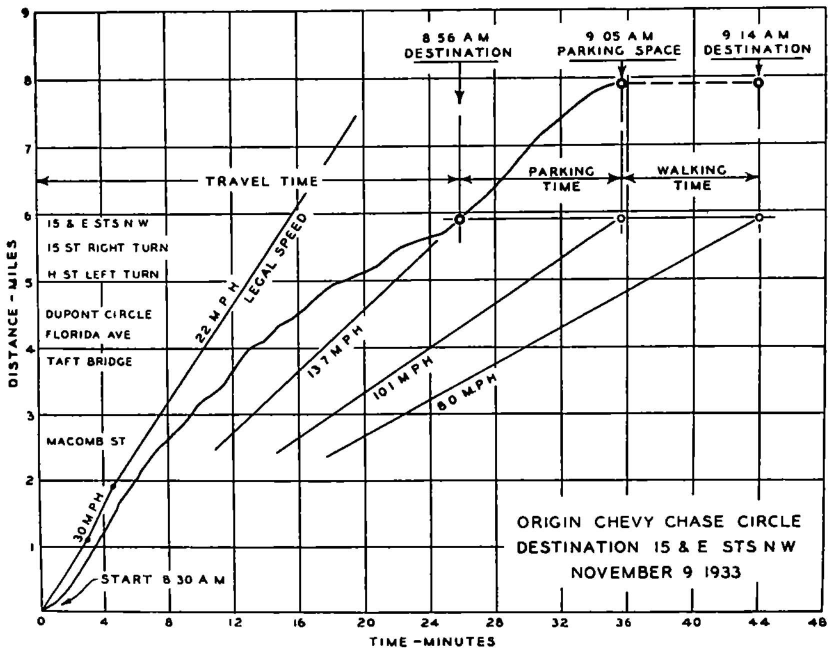


Figure 8. Effect of Lack of Parking Facilities upon Trip Time

Figure 8 is for a trip down Connecticut Avenue from Chevy Chase Circle to the Commerce Building Here the speeds are 13 7 miles per hour to destination, 10 1 to parking space and 8 0 miles per hour to the office

The average times of the three runs are 14 2 miles to destination, 10 8 miles per hour to parking space, and 8 5 as the final average after the walk back to the destination after parking

The last word in motor car design, so accurately streamlined that hardly a particle of dust will accumulate on its fenders and body surface, is shown in Figure 9

Figure 10 shows the first steamboat, the Clermont, which sailed up the Hudson in 1807

After a century of progress which saw the birth and development of the telegraph, the telephone, the motion picture, the radio, the talking picture, and the airplane, this automobile, the acme of ingenuity and skill, which on a road surface serviceable for high speeds, is capable in rural transport of bettering with ease and comfort standard train schedules, when used in urban transport is slowed down, first, by legal regulations, second, by traffic congestion, and third, by poor terminal facilities, until the average speed of travel including the walk back to

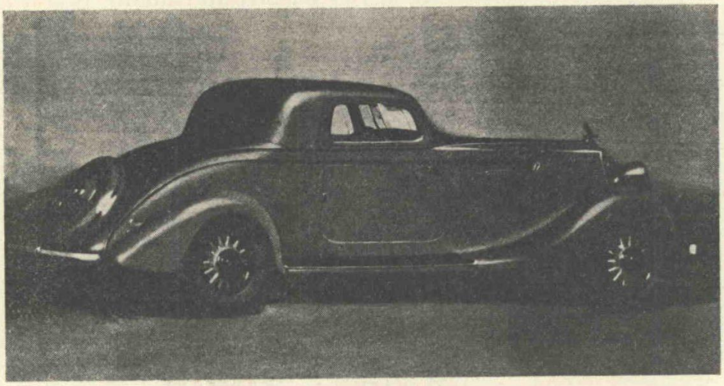


Figure 9

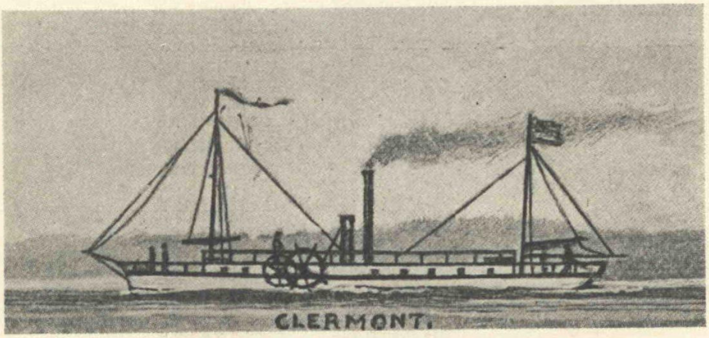


Figure 10

the office, store or shop becomes just a few miles per hour greater than that of Robert Fulton's first steamboat on the Hudson River 124 years ago.

This is no reflection on the traffic engineer and no criticism of the traffic regulations and control mechanism. His job has been to make one parking space serve from 5 to 10 cars. In Washington's business section, for instance, his job has been to accommodate a possible 100,000 cars daily in curb space capable of accommodating only 10,000. And he has done his job well. But you can not park your car on a regulation.

Thus we reach the natural conclusion aptly stated in the "Report of the Automobile Parking Committee of Washington," published under the chairmanship of Colonel U S Grant, 3rd, "that additional provision must be made (for parking space) if this form of transportation is to develop " Thus parking space instead of financial ability becomes the true index of the saturation point of urban car ownership

The consideration of terminal facilities concerns not how much the car owner is willing to pay for such service but instead the expenditure justified by sound economics to increase the speed, comfort and safety of the entire traveling public by removing parked cars from the highly

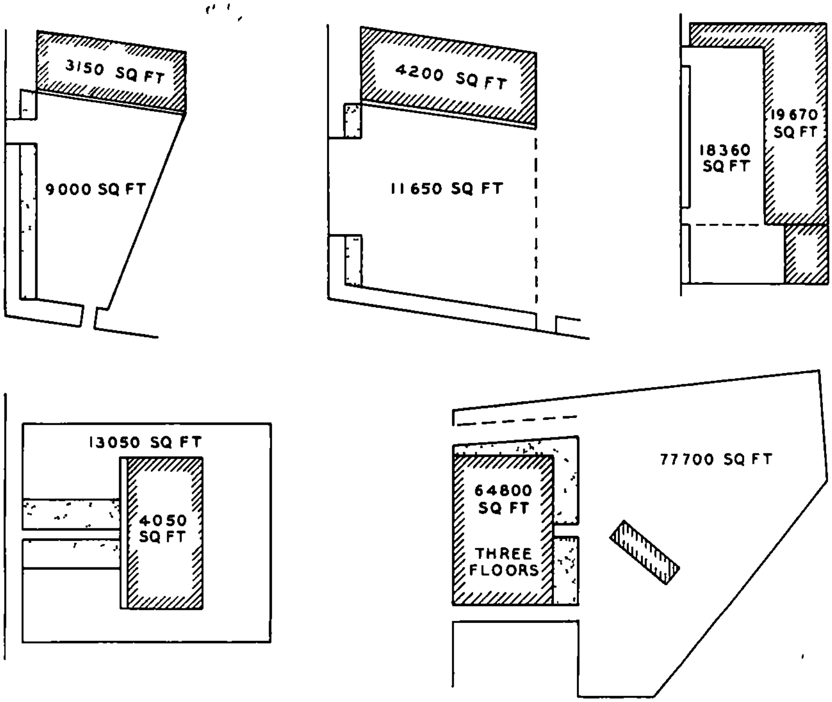


Figure 11 Relative Store Area and Parking Area of Typical Industrial Establishments

expensive street surfaces to off curb parking areas with less expensive surfacing

Parking space is not merely a personal convenience for car operators, but an integral part of economic motor transport Consequently, expenditures justified for its provision can be computed on the same basis as expenditures for increased road width, smoother riding surfaces, reduced curvature, greater sight distance, wider bridges, and better alignment Included also is the effect of adequate terminal facilities on putting more cars into service with their purchase price going to pay the cost of wages and materials of manufacture and their operating costs for tires, gas, oil, repairs and insurance going to increase the general industry and prosperity of the nation.

Whether an individual is willing to pay 10, 20 or 50 cents per day for parking facilities is subordinate to the placing in service of additional

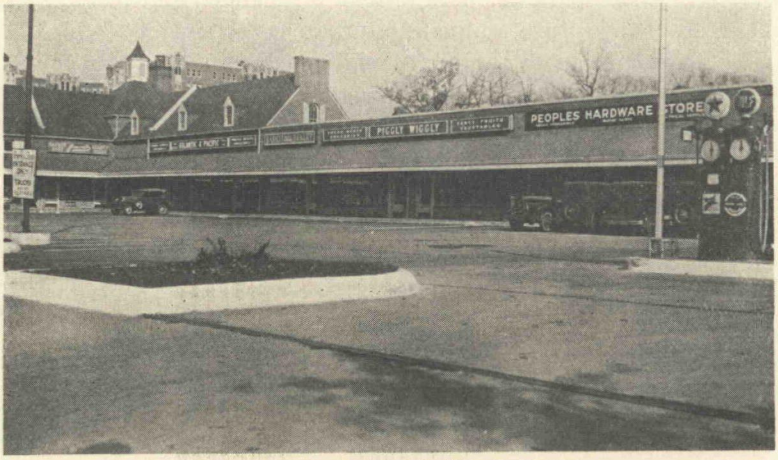


Figure 12. Off Curb Parking Space

Figure 13. Parking Area

units of transportation which contribute considerably to the commerce of the community and the nation.

Savings which accrue from provision of adequate parking facilities might be computed from several bases as follows:

1. The difference in value of parking lot and street surface.
2. The increase in speed of travel of moving traffic due to wider roadways.

3. The reduced operating costs due to shorter distance of travel.
4. Saving of time when necessity for hunting parking space is eliminated.

Thus, M. O. Eldridge in "Cost of Parking," Engineering News-Record, February 9, 1933, shows that the carrying charges for parking

Figure 14. Neighborhood Store Parking

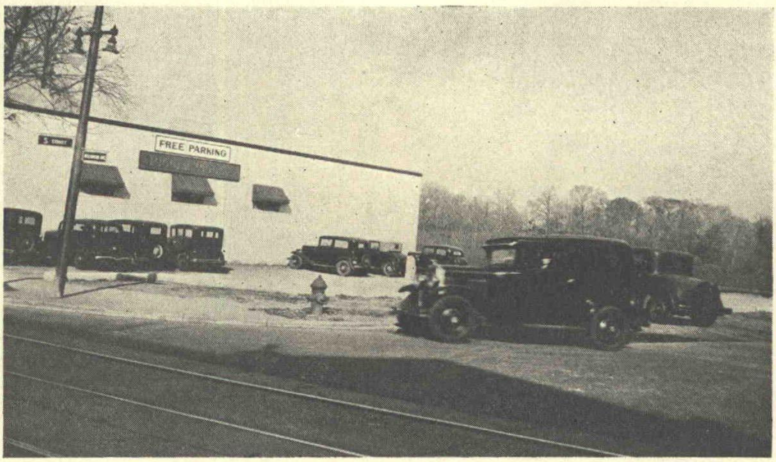


Figure 15. Neighborhood Store Parking

space for one car on Fifth Avenue, New York, about 156 square feet, amounts to $142 per month, or about ten times the cost of parking in the modern garage.

On the assumption that parking space can be covered with cheaper surfacing than street surfaces, additional savings can be estimated.

If this difference amounts to $2.00 per square yard, the capital savings for 400 cars occupying two miles of curb space and requiring about 7,000 square yards of off curb space would amount to about $14,000.

On the basis of 20,000 cars per day, which can be taken as the traffic on a main artery like Pennsylvania Avenue, the savings due to increase of speed of travel due to parking elimination could be enormous. If this increase of speed were only five miles per hour, the time saving in a year of 300 days would amount to 6,000,000 vehicle minutes per mile annually.

At one cent per vehicle minute, the saving proposed by Sigvold Johannesson in his book "Highway Economics," the annual return

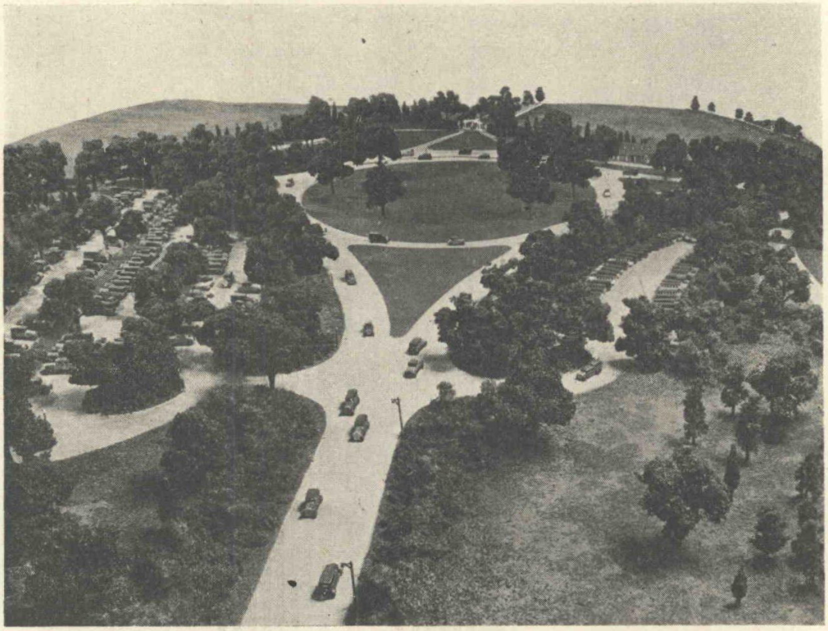


Figure 16. Parking Areas at Mount Vernon

becomes $60,000 per mile annually or the interest at five per cent on $1,200,000.

The average distance from destination to parking space in the three runs is 1.65 miles. At 400 cars per day, 300 days per year, this becomes 198,000 miles which, at five cents per mile, cost $9,900 per year. This capitalized at five per cent is equal to $198,000.

I venture no estimate of the value of the time which must be spent in walking back to the destination. This is one of the "musts" however, which diminish the intangible returns which have made the highway and motor satisfy a national transportation need not furnished by any other type of transportation. All such necessities introduce into motor transport the type of irksome travel restrictions which motor transport is designed to eliminate Exclusive of this item and also of any carrying charge on the valuation of the street area, the savings accruing from placing 400 cars per day in off curb space per mile of street carrying 20,000 vehicles per day becomes

$$14{,}000 + 1{,}200{,}000 + 198{,}000 = \$1{,}412{,}000$$

The cost of providing the $1\frac{1}{2}$ to 2 acres of parking space for the 400 cars subtracted from $1,412,000 represents the profit due to furnishing off curb space for parking the cars now occupying one mile of street Thus, like the old argument for good roads "we pay for parking space whether we have it or not and it costs more not to have it than to have it"

That this is recognized by some of our old conservative business organizations is shown by Figures 11 to 16 They show what is being done toward providing the 400 acres with attendants or the 800 acres without attendants required for Washington's 100,000 vehicles

Along with the parking facilities we might include also other fields of change in physical structure which are justified part by the tangible and part by intangible economics Such problems include the following

1 *Development of design of sidings for expediting traffic on two-lane traffic roads* With mixed traffic of appreciable amount on two-lane traffic roads, the speed of traffic is practically that of the slower moving vehicles To increase the width of all such roads as occasion demands is a physical as well as a financial impossibility However, single lane sidings placed at proper intervals to furnish a three-lane road, especially on upgrades where the faster units can pass the slow moving traffic, will accomplish much toward speeding up traffic

2 *Development of anti-skid high speed road surfaces* The annual casulty lists of about 30,000 killed, 85,000 permanently and 945,000 temporarily injured in highway transportation is contributed to largely by skidding automobiles This is reason enough for studying the mechanics of skidding and preventive measures in both construction and maintenance Here it may be necessary to increase the cost of operation due to greater tire wear in order to reduce expensive loss of life

3 *Development of methods for segregating pedestrians from auto traffic especially in urban districts* This is to make a further cut in the casulty list and to speed up traffic as well because, according to estimate, pedestrians are concerned in about two-thirds of the traffic accidents

For years, attempts have been made to legislate pedestrians into certain locations for crossing streets Even at these designated locations pedestrians have to be more and more alert and be able to look in three or four directions at the same time to prevent being run down This is not only a city problem, for the bypassing of small towns is but a temporary expedient which lasts only long enough for the gas station, the barbecue, drug store and the rest of the town to move to the new location

The only logical remedy is, through research, to devise the means for gradually preventing the pedestrians from using the street roadway, thus following the example of Radburn, New Jersey The design of underpasses or overpasses and the selection of their proper location for accomplishing this, combined with the design of accompanying parking areas, loading platforms and corresponding methods of traffic control, furnish an almost unlimited field for improvement in highway transportation

The railroad engineer has been solving a similar problem for the past half century Prohibitive restrictions made increasingly necessary the separation of tracks from the street grades on which rail transportation originally developed This called for new routes and often new terminals as well Terminals with through routes for highway transportation can follow the same general principles of design and economic justification

If suggestions such as these are considered impractical and the highway field fully developed and, if in the city the curb line is considered the ultimate barrier beyond which the highway builder may not pass, the field of future motor transport development is indeed limited If, on the other hand, we are willing to recognize that scientific inquiry into new fields of structural design can promote the comfort, speed and safety of highway transportation, the road builder has an unlimited field

Several years ago, at a Highway Research Board dinner, Colonel Chevalier asked the question—What will the highway engineer do when he is through building highways? The foregoing might serve as a reply for it suggests the almost unlimited task required to remold the urban structure to accommodate modern motor transport requirements instead of restricting motor transport development to the urban facilities of horse and carriage days And in this remolding the aim is profits, not costs.

DISCUSSION ON INTANGIBLE ECONOMICS

Mr J Rowland Bibbins, *Washington, D C* On this particular phase of economics there is now much disagreement in Washington Mr. Hogentogler has introduced the problem of remodeling the entire urban structure Let us look at this problem of the community as a whole, urban and suburban, regardless of political boundaries, draw a circle around it, and work out the totalized balance sheet of the distribution of costs, the benefits and problems of development Those of us who have been studying urban districts intensively recognize the growing blight that has so much affected internal land values and the housing of people This cannot be discussed now, but it is an outstanding problem of economic relatives Rapid transit has contributed to the blight and so have express highways It has opened up a large subject which I hope will go on the programs of succeeding meetings.

REPORT OF COMMITTEE ON DESIGN

A T Goldbeck, *Chairman*

Chairman Highway Research Board

CRACK CONTROL IN CONCRETE BASES FOR SHEET ASPHALT PAVEMENT

By John V Keily

Materials Engineer, Rhode Island State Board of Public Works

SYNOPSIS

This is a progress report on an experimental road built in 1931 A 46 foot pavement, constructed in two lanes, has a plain concrete base on one side and experimental sections on the other These latter sections contain many methods for using wire mesh reinforcement, expansion joints and dummy joints to control cracking in a six inch concrete base Indications of success or failure of the various designs after two years of traffic and exposure are reported and recommendations made for future work based on results obtained to date

Numerous sheet asphalt pavements of conventional design, on six inch concrete bases in suburban sections of Rhode Island have developed unsightly cracks after relatively short periods of use These cracks which occur first in the base carry up through the binder and surface courses and their initial irregularities are magnified by the interlock of the $\frac{3}{4}$ inch stone in the binder course Finally they appear on the surface and wander aimlessly across the pavements in diagonals, forks or zig-zags until they reach the curb or a longitudinal joint In an effort to improve this feature of the pavement the Rhode Island Board of Public Roads, in cooperation with the Committee or Design of the Highway Research Board, set apart a project on Newport Avenue in Pawtucket for experimental purposes in September, 1931 The road was approximately 46 feet wide and was built in two sections, one lane, 16 to 20 feet wide at places including a car-track, was poured first, using ordinary methods of construction and the balance of the road 26 to 30 feet wide was selected for the construction of the experimental sections The road was built on practically an ideal subgrade of sand and light gravel and on rather flat grades No particular attempt was made to insure a slip joint between the experimental sections and the curb or the adjacent parallel lane of pavement It was thought that, if anchorage should occur on these two sides of the slab, it would increase the efficiency of the dummy joints and the included reinforcing.

A 1:2 5:5 mixture of Dragon cement, washed sand and washed crushed gravel with a 5 to 6 inch slump was used In the sections where reinforcement was used, the base was roughly screeded to 2 5 inches below the finished grade and the mesh placed as called for, with the remaining concrete deposited to final elevation and finished with the backs of shovels Three cylinders and three beams were made every second day during concrete runs and were broken at the ages of 7 days, 28 days, and 6 months The average compressive strengths were 1475, 2260 and 2883 lbs per sq in respectively while the beams gave moduli of rupture of 428, 595 and 645 lbs. per sq in at corresponding ages

A record was kept of all cracks appearing in the base before the surface courses were laid. A period of approximately seven days was available for these observations and, in most instances to date, cracks in the asphalt pavement have developed over cracks which appeared in the base before the top was placed It was rather difficult to see whether cracking of the base had really taken place at dummy joints It was observed, however, that there were no other cracks in these sections If contraction had taken place, it is quite possible that cracks may have been present at the bottom of the 1 5 inch slot without being apparent to the observers A movement at these joints at a later date is expected to show in the finished pavement

In addition to the sheet asphalt construction, a 1400 foot section was constructed with a modified sheet asphalt wearing course in which several of the experimental base sections were repeated The change in wearing course was made by adding 30 per cent of ½ inch crushed stone (ranging in size from ¼ to ¾ inch) to the standard sheet asphalt mixture In all asphalt construction a ten-ton three-wheel roller was used for initial compaction followed by a six-ton tandem roller operating diagonally

TYPICAL BASE SECTIONS

Type A· This is a typical plain concrete base, six inches thick and easy to place, with transverse joints made only at the end of a day's run

Type B A six inch concrete base with ½ inch expansion joints placed 30 feet apart and ⅔ filled with 85–100 penetration asphalt A thin galvanized sheet was placed over each expansion joint to prevent the binder from entering and destroying freedom of movement at these joints

Type C· This section has a plain six inch concrete base with dummy joints (1 5 in deep) at short intervals designed to control contraction cracks

Type D The same six inch concrete base with welded wire reinforcement placed 2 5 inches below the surface and between dummy joints spaced approximately 12 feet apart A two inch gap between sheets of mesh at dummy joints was planned The mesh was used to hold sections together between dummy joints

Type E The same six inch concrete base containing different weights of wire mesh reinforcement placed 2 5 inches below the concrete surface but with free ends of mesh lapping four inches across dummy joints which were placed at intervals of 24 feet This type was similar to Type D except that the shearing value of the dummy joints was increased by the addition of the steel wires crossing them

Type F A six inch concrete base with continuous wire mesh reinforcement Originally it was planned to place dummy joints 21 feet apart in a portion of this section to assist the continuous mesh to break down at these intervals but the dummy joints were detailed over the locked ends of the reinforcement so that this feature was practically eliminated In effect this section has continuous reinforcement thoughout its length This was evidenced in the field by the fact that, when the cracks appeared, they did not occur over the dummy joints as made

OBSERVATIONS AT THE END OF TWO YEARS

The general observations that can be made at this time serve mainly as a basis for further experimental work

It should be kept in mind that these observations are limited to a pavement with a fairly strong concrete base which is expected to crack and that these experimental sections were designed to control these cracks rather than eliminate them

GENERAL INDICATIONS, WHICH APPLY TO BOTH PLAIN AND REINFORCED SECTIONS

1 Numerous cracks in the base appeared within seven days after the concrete was poured and before the asphalt surface was placed

2 Most of the cracks now visible in the asphalt surface occur over cracks previously observed in the base

3 Several cracks, in the wearing course, noted recently, do not reoch the curb by some four to seven feet All were over old cracks or joints in the base This seems to indicate that incipient cracking of the surface course may be accentuated by traffic whereas the area near the curb, under little or no traffic, may resist cracking even over relatively wide contraction joints

4 The above observation includes cracks over two or three dummy joints where cracks may have existed originally at the bottom of the 1 5 inch slots in the base, but were not apparent to observers before the asphalt surface was laid

5 Cracks, occurring within short distances in adjacent lanes of this pavement, generally connected up, at the expense of rather unsightly irregular longitudinal cracks in the surface

6. Irregularity of cracks starting in the base has been accentuated

before reaching the surface probably by the interlocking action of the ¾ inch binder course Even cracks above straight dummy joints are more or less uneven upon reaching the surface

7 Expansion joints designed with cover plates, have generally produced wide and unsightly cracks without adding to the structural strength of the pavement Michigan experiments* indicate that without some mechanical means of transferring loads across these joints, the pavement has been actually weakened and that the use of a cover plate, to prevent the binder from entering these expansion joints, has resulted in wide cracks on both sides of this plate The behavior of plain sections in this pavement and many years of experience with similar construction in Rhode Island indicates that there is no need for expansion joints in this type of pavement when a base with not stronger than a 1:2 5.5 mix is used We have no record of blow-ups to date

INDICATIONS WHICH APPLY TO DEFINITE TYPICAL EXPERIMENTAL SECTIONS AT THE END OF TWO YEARS

Type A Plain concrete base without joints and exclusive of area affected by car-tracks

(a) Plain concrete base sections have averaged cracks about 60 feet apart in two years although individual lengths have varied from 13 to 90 feet

(b) Practically all of the base cracks were reproduced in this asphalt wearing course at the end of two years, although one or two exceptions have been noted

(c) Individual cracks were invariably transverse, but irregular in direction and not always at right angles to curb

Type B Plain concrete base with ½ inch expansion joints, 30 feet apart

(a) Joints in this section most consistently produced cracks where designed because all but one joint functioned This seems to indicate that definite planes of cleavage should be constructed in the base The last joint did not act because it was locked in place by a return of the base into a street intersection

(b) The objections to this design are slowness of installation, the development of double cracks in the asphalt surface caused by cover plates over these joints, and the lack of load transfer across straight joints

(c) Although the original expansion joints in the base were constructed to the curb, the only cracks visible on the surface now are narrow cracks ⅛ inch wide appearing in the heavier traveled part of the street These cracks have not appeared within six feet of the granite curb

* Investigation of the Shear Resistance of Cracks, A C Benkelman *Engineering News-Record*, August 24, 1933

Type C· Plain concrete base with dummy joints, 1 5 inch deep

(a) Dummy joints made 1 5 inch deep and spaced from 12 to 21 feet apart, do not *always* appear to function as contraction joints The tension area of concrete below the joint is sufficiently strong to hold several sections together even at early ages Assuming a 500 lb tensile strength in the concrete, computations show that it is possible to develop 135 tons of resistance to breakage in a ten foot width of this base

(b) Cores have proven that all cracks in asphalt surface in this section have occurred over dummy joints in the base, but only every third or fourth dummy joint at a distance of 45 feet or more has produced a crack

(c) There is better uniformity in alignment of cracks appearing over dummy joints in both plain and reinforced sections and an absence of the diagonals, forks, and zig-zags that were observed where no joints were provided Cores have shown that minor offsets up to three inches from the straight cracks at dummy joints, are produced by the interlocking effect of the $\frac{3}{4}$ inch binder course

(d) Individual slabs in the base between dummy joints all appear to be intact No cracks in the asphalt surface developed between dummy joints in the base

Type D Concrete base with mesh reinforcement in each 12 foot section separated by dummy joints in the two inch space between sheets of mesh

(a) The tension area below dummy joints in this section caused seven sections of base to hold together as did those noted under the plain concrete section with dummy joints One single crack in this area, with 84 feet of unbroken surface on either side, was $\frac{1}{4}$ inch wide

(b) A core showed that this single crack in the base occurred at a dummy joint, although the break in the surface offset about three inches from the dummy joint below due to the interlock of the binder course

Type E Reinforced concrete base with ends of mesh lapped, with and without dummy joints

(a) Dummy joints, crossed with lapped ends of reinforcement, have had little chance to show their possible superiority over plain joints, because long sections of base including this feature have held together as was noted in the plain concrete sections

(b) Unbroken sections were, however, much longer in this area running to a maximum length of 160 feet between cracks, although dummy joints were installed at the end of every third sheet of reinforcement or about 23 feet apart The minimum length of slab in this section is 60 feet

(c) There is no choice in regard to weight of mesh reinforcement

Type F· Reinforced concrete base with ends of mesh locked with and without dummy joints

(a) As previously mentioned, dummy joints in this section were detailed and placed over the locked ends of reinforcement and, therefore, failed to act This section therefore can be regarded as one of continuous reinforcement

(b) The theory that continuous reinforcement might break the base up into minute sections, evenly distributed, was not borne out in these trials Instead, the reinforcement held the base intact in relatively long sections With 32 lb mesh the slabs between cracks averaged 75 to 86 feet in length, with a minimum of 50 feet and maximum of 100 feet of unbroken asphalt top With 42 lb mesh the slabs between cracks averaged 100 to 102 feet, with 60 feet minimum and 165 feet maximum of unbroken lengths of surface

(c) A few base cracks from 20 to 30 feet apart, which developed before the top was placed, have not appeared in the surface. This may be due to the reinforcement holding the adjacent sections close together

(d) No particular uniformity in spacing or alignment of cracks is apparent in these sections

(e) Cores have indicated that the steel has broken at least at the end of the longer sections

(f) Cores have indicated that some definite method of locking sheets of mesh should be devised that would prevent sheets from separating when walked upon by workmen, thus destroying the continuity in reinforcement desired

SUGGESTIONS FOR FUTURE EXPERIMENTAL WORK

Based on our experience with these experimental sections, the investigators working on this project make the following suggestions for trial in future experimental projects of similar nature.

1 That dummy joints, as designed in this project, be replaced by light, deformed-steel contraction joints spiked to the subgrade and left in place These joints should be just sufficiently strong to form an effective key between sections and extend at least one inch into the binder course to straighten out surface cracks This design would eliminate the tension areas in concrete at dummy joints and provide for positive load transfer across joints where a strong base is needed

2 That in areas where an excellent subgrade is available and load transfer across joints is not of extreme importance, dummy joints made to a greater depth or at least one third of the depth of the pavement be tried. A temporary steel strip extending through the binder course could be used to keep joints in sheet asphalt surface straight and could be removed after the binder is rolled

3 That thin premoulded expansion joint material or at least heavy roofing felt be placed between the concrete base and the face of curb to prevent anchorage and insure functioning of these contraction joints.

4 That these transverse contraction joints be placed at intervals of not more than 20 feet longitudinally to form relatively fine cracks which if reproduced in the asphalt surface will heal under summer traffic and require little or no maintenance in winter

5 That the same light, deformed metal strips be used for longitudinal joints to transfer loads between lanes These forms will not need to extend into the binder course if this surface course is held back from the edge of base during construction

6 That contraction joints in parallel lanes be placed opposite one another

7 That in the design of contraction joints for a base such features as manholes or the return of the base into side streets, be taken into consideration These structures give definite anchorage and might prevent the operation of contraction joints unless provided for in the design

8 That expansion joints be eliminated from all concrete base construction used under asphalt pavements

9 That wire mesh be included in sections between contraction joints if the subgrade is poor

Although this paper has mentioned many cracks in this pavement, the investigators really had to look for them and we would emphasize here that this pavement as a whole is in excellent condition today It was only with a hope that we could come nearer to perfection in building this type of surface and eliminate a certain amount of maintenance that this experimental work was undertaken We hope that by stressing certain imperfections we have not undermined your confidence in a type of pavement that is giving fine service in thousands of communities today.

AN EXPERIMENTAL ROAD OF CEMENT BOUND MACADAM[1]

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[Condensed[2]]

Resumption of construction of cement bound macadam after a dormant period of nearly two decades, disclosed a lack of knowledge of

[1] This report was presented through the Committee on Design as information on the development of an interesting type of road construction No conclusions or recommendations are offered by the Committee

[2] A more detailed report of this investigation was published in Engineering News-Record, February 15, 1934 Volume 7, page 230

material requirements, best construction methods and data on which designs and estimates could be based. An experimental road on original soil near Elmhurst, Illinois, was built by the Portland Cement Association to provide this information.

The test road included 81 sections constructed with three sizes of limestone, three sizes of gravel and one size of slag coarse aggregate. These coarse aggregates were penetrated with grout proportioned 1:1, $1{:}1\frac{1}{2}$, 1:2, $1{:}2\frac{1}{2}$, $1{:}2\frac{3}{4}$ by weight, made of coarse, medium and fine sands and having varying amounts of mixing water. Groups of sections were compacted by rolling with 5.8 ton and 3.3 ton tandem rollers, by vibratory equipment and by hand tamping. The details of the test sections are given in Table III.

Figure 1. Flow characteristics of grout were determined by tests with a "flow cone."

PENETRATION OF COARSE AGGREGATE

A study of the requirements for uniform and full penetration of coarse aggregate by the grout was one of the more important features of the investigation. Obviously penetration is influenced by the fluidity of the grout and the sizes of the spaces between the coarse aggregate particles. Successful penetration was found to depend primarily on grout which would flow freely and without segregation into the coarse aggregate.

In order to study the fluidity of grout it was first necessary to develop some apparatus which would measure the flow under variable conditions. A "Flow cone" (Figure 1), consisting essentially of a funnel to the bottom of which is fitted a ½ by 1½ inch tube, was found to measure satisfactorily and consistently the fluidity of grouts as well as the approximate conditions of flow under actual working conditions The flow tests showed that the mix, size and grading of the sand and the amount of mixing water affect the fluidity It can be seen (Figure 2)

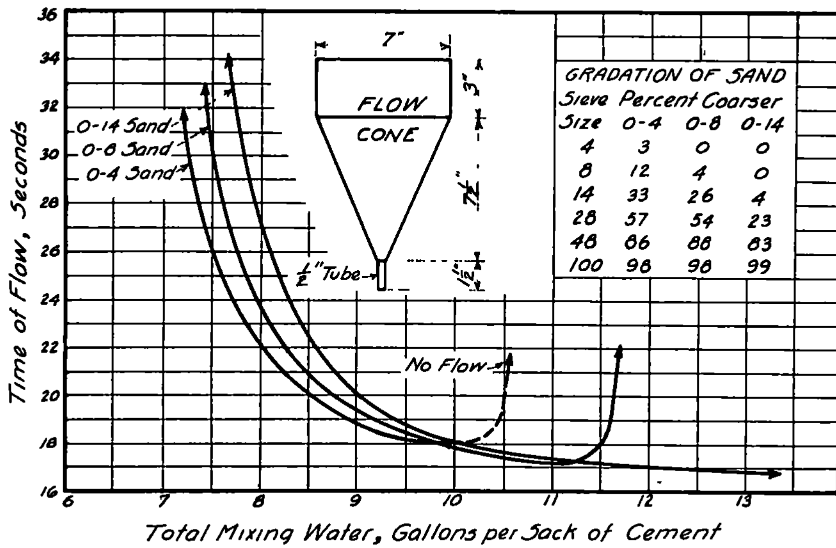

Figure 2 Flow Curves for Grouts Made of Coarse, Medium and Fine Sands. Mix 1 2 by weight

that the grout becomes more fluid (measured by decreasing flow time) as the amount of mixing water increases For coarse and medium sand this only holds true up to the point of segregation where additional water causes reduced flow time or stoppage Objectionable bleeding of free water occurs at water contents below this amount

TABLE I

Coarse Aggregate, Inches	Flow, seconds	Sands, Screen Numbers
1½–3	23–25	0–4, 0–8, 0–14
1 –2	20–22	0–8, 0–14
½–1½	17–19	0–14

The point at which this objectionable bleeding occurs in a grout made with any given sand and mix definitely limits the fluidity which can safely be attained with that sand and mix Experience having indicated the fluidity necessary to insure satisfactory flow of grout through coarse aggregates of the different sizes, it was possible to select sand with which the required rates of flow could be safely reached

It can readily be imagined what would happen should a coarse sand grout be used with small sizes of coarse aggregate If the water content were kept low enough to prevent segregation, the grout would not be sufficiently liquid to flow through this coarse aggregate although it might be quite satisfactory for coarse aggregates larger in size If the water content were increased, the coarse sand particles would separate and be deposited in the spaces in the topmost layer of coarse aggregate, and only the most liquid portion of the grout could filter through This would result in a honeycombed condition in part of the layer of coarse aggregate

Coarse sand grouts are therefore restricted to use with large aggregates, and fine sand grouts must be used with small coarse aggregates, requiring a more liquid grout Table I shows the rather definite limits fixed by the investigation for the fluidity and hence the sand sizes of 1:2 grouts which can be used to penetrate different sizes of coarse aggregate

COARSE AGGREGATES

Gravel, limestone and crushed slag were used in the tests

Limestone having a wear loss of 8 per cent and a toughness as low as 10 did not break up under excessive rolling This is considerably below what was originally thought possible and materially widens the range of useable aggregates Gravel aggregates containing more than about 5 per cent soft particles are objectionable

Large size coarse aggregates can be penetrated with grout made of a wider range of sand sizes but are more difficult to spread and require more grout than small sized aggregates Also pavements made of large aggregates are more difficult to finish than those made of small aggregate The optimum sizes range between a $\frac{3}{4}$ to 2 inch size and a $1\frac{1}{2}$ to $2\frac{1}{2}$ inch size The upper range should be used for slag to compensate for crushing of particles where compacted by rolling

COMPACTION METHODS

Rolling with the 5 8 ton tandem roller resulted in greatest compaction, highest strength and use of least grout per inch of volume, but caused greatest loss of coarse aggregate through pushing into the subgrade Rolling with a 3 3 ton roller gave appreciably lower strength and less compaction than with the 5 8 ton roller although subgrade losses were but little less

Vibration produced strength just below that obtained with a 5 8 ton roller although with materially less compaction, except in the case of gravel aggregate, hence, more grout per unit volume was required The loss of coarse aggregate into the subgrade was negligible

Compaction by hand tamping gave lowest strength and least consolidation but the loss of aggregate into the subgrade was negligible The success of this method opens the field of cement bound macadam to small jobs where other compacting equipment is not available or accessible

Compaction, except by vibration, during grouting operations is of little or no aid to penetration Volume of coarse aggregate and grout consumption was more effectively reduced by compaction prior to grouting than by compaction after grouting Initial compaction also reduced the stone loss and made it easier to judge grout requirements and to obtain a good surface finish

Final compaction should be done at a suitable interval of time after grouting in order to consolidate the voids left in the grout by escaping free water, thus causing the quality and strength to correspond with the water cement ratio of the finished pavement Analyses showed that grouts mixed with 7 5 gallons of water per sack of cement retain 5 3 gallons per sack at the time of final compaction This accounts for the high strengths obtained

TABLE II

AVERAGE STRENGTHS—CEMENT BOUND MACADAM—1 2 GROUT, MEDIUM SAND

Compaction by	Modulus of Rupture, Lb per sq in		Compressive Strength, Lb per sq in
	7 Day	28 Day	28 Day
5 8 ton roller	567	687	3993
Vibration	528	638	3831
3 3 ton roller	465	643	4352
Hand	388	567	3892

The test specimens were taken from the finished pavement

STRENGTHS

Pavement built with 1.2 grout of medium (0–8) sand developed the average strengths shown in Table II The 7 day bending tests and 28 day compression tests for all sections are given in Table III The test specimens (beams and cores) were taken from the finished pavement

FINISHING METHODS

Following compaction whether by rolling or vibration, the pavement was leveled with a longitudinal tamping template It was found to be very effective for this purpose and is largely responsible for the good riding qualities obtained This was followed by smoothing with a long handled float and burlap drag Final finish was obtained by brooming transversely with a fibre broom

SUMMARY

The principal conclusions and recommendations derived from the investigation are as follows

Pertaining to subgrade Loss of coarse aggregate into soft subgrades can be overcome with subgrade treatments consisting of relatively thin layers of coarse material, thoroughly rolled into the grade These are more effective than finer granular material mulched into grade

Pertaining to coarse aggregate Gravel, crushed limestone and slag are satisfactory for use in cement bound macadam construction

Large sizes of coarse aggregate are easily penetrated but require more grout and are more difficult to spread than small size aggregate Pavement made with large coarse aggregate is more difficult to finish than that made with small coarse aggregate

Coarse aggregate ranging in size between $\frac{3}{4}$ to 2 inch and $1\frac{1}{2}$ to $2\frac{1}{2}$ inch is the optimum for cement bound macadam

The loss of coarse aggregate into the subgrade, under comparable conditions, is less for gravel than for crushed stone or slag, and for the smaller sizes of the same kind of aggregate

Pertaining to grout The "flow cone" developed during these tests furnishes a practical method for measuring and controlling grout fluidities

To avoid objectionable bleeding, grout proportioned 1:2 by weight and made of the coarse (0–4) sand should be limited to a fluidity corresponding to a flow time of 21 seconds Similar grouts made of the medium sand (0–8) should be limited to a flow of 18 5 seconds If more rapid flow is desired, the fine (0–14) sand should be used

To avoid objectionable bleeding, grout proportioned $1:2\frac{1}{2}$ by weight, made of the medium sand should be limited to a flow of 19 5 seconds and the $1:2\frac{3}{4}$ mix to 20 5 seconds

Grouts proportioned 1:2 by weight made of the medium (0–8) sand produced pavement of somewhat higher strengths than grouts of the same mix made of the coarse (0–4) or fine (0–14) sand

No more mixing water should be used than that needed to produce the fluidity required for penetration

A volume of grout averaging 17 per cent in excess of the volume of voids in the coarse aggregate of the fully grouted pavement is required to compensate for the loss of grout and shrinkage in grout volume due to escaping water

Pertaining to compaction Cement bound macadam can be satisfactorily compacted and finished by rolling, by vibration, or by hand tamping

Rolling with the 5 8 ton roller gives the greatest compaction and highest strength It also results in greater loss of coarse aggregate into subgrades

Vibration gives strength just less than that obtained with the 5 8 ton roller, materially less compaction, except with gravel aggregate, and less subgrade loss.

Rolling with 3 3 ton roller gives lower strengths and appreciably less

TABLE III

CHARACTERISTICS OF CEMENT BOUND MACADAM TEST SECTIONS—ELMHURST EXPERIMENTAL ROAD

Section Number	Coarse Aggregate				Grouted Depth, Inches	Strength Tests Lb per Sq In		Materials			Vibratory Equipment	$\frac{w}{c}$ Gallons per Sack	Flow Time, Seconds	Grout Mix
	Kind	Size	Loose Depth, Inches	Final Depth, Inches		7 Day Modulus of Rupture	28 Day Compression	Cement Sacks per Cu Yd	Sand Cu Yd per Cu Yd	Coarse Agg Cu Yd per Cu Yd				
Project X—Preliminary investigation of equipment, compaction, subgrade loss, grout consistency, penetration, organization of crew														
00	Limestone	1 to 2	8 1	7 2	6 0	No beams	4,100	4 96	0 312	1 333				
0	Limestone	1 to 2	7 9	6 5	6 5	No beams	5,000	4 58	0 288	1 212				
Project A—Constants grout mix, 0–8 sand, $\frac{w}{c}$, flow, compaction by 5 8 ton tandem roller Variables type and size of coarse aggregate, no initial compaction on gravel, twice over on stone and slag, final compaction 3 to 8 times over as required														
1	Limestone	1 5 to 3	7 8	7 2	6 2	840	5,000	4 71	0 296	1 256		7 5	21 8	1 2
2	Limestone	1 to 2	7 9	6 8	5 9	625	4,350	4 37	0 273	1 338		7 5	21 8	1 2
3	Limestone	0 5 to 1 5	8 2	6 8	3 0	No test	4,600	Penetration Failure, Section Removed, see 3R				7 5	21 8	1 2
4	Gravel	0 75 to 2	8 3	7 5	6 7	594	3,950	4 34	0 272	1 233		7 5	21 8	1 2
5	Gravel	0 5 to 1 5	8 1	7 5	5 5	No Test	4,580	Penetration Failure, Section Removed, see 5R				7 5	21 8	1 2
6	Slag	0 75 to 2	8 0	6 8	3 6	472	3,600	Penetration Failure, Section Removed, see 6R				7 5	21 8	1 2
Project B—Constants Grout mix, 0–8 sand, $\frac{w}{c}$, flow, compaction by 5 8 ton tandem roller Variables type and size of coarse aggregate; no initial compaction on gravel, twice over on stone and slag, final compaction 4 to 5 times over as required														
7	Limestone	1 5 to 3	8 0	7 2	5 7	655	3,850	5 10	0 320	1 399		8 5	18 6	1 2
8	Limestone	1 to 2	7 7	7 0	5 5	607	4,530	4 64	0 290	1 386		8 5	18 6	1·2
9	Limestone	0 5 to 1 5	7 9	7 2	6 2	628	3,100	4 32	0 271	1 263		8 5	18 6	1 2
10	Gravel	0 75 to 2	7 4	7 0	5 6	523	3,880	5 17	0 325	1 310		8 5	18 6	1 2
11	Gravel	0 5 to 1 5	7 3	7 1	6 2	No Test	3,140	3 66	0 230	1 178		8 5	18 6	1 2
12	Slag	0 75 to 2	8 2	7 3	6 6	512	3,670	4 28	0 269	1 242		8 5	18 6	1.2

Project C—Constants Grout mix, 0–4 sand, $\frac{w}{c}$, flow, compaction by 5 8 ton tandem roller Variables type and size of coarse aggregate, no initial compaction on gravel, twice over on stone and slag, final compaction 4 to 6 times over as required														
13	Limestone	1 5 to 3	7 9	7 2	5 7	535	4,407	5 11	0 312	1 377		7 25	20 8	1 2
14	Limestone	1 to 2	8 5	7 3	6 2	524	4,253	4 53	0 272	1 368		7 25	20 8	1 2
15	Slag	0 75 to 2	8 2	6 9	4 2	No Test	3,390	Penetration Failure, Section Removed, see 15R				7 25	21, 18	1 2
16	Gravel 40% crushed	0 75 to 2	7 7	7 1	6 3	684	3,443	4 25	0 260	1 219		7 25	20 8	1 2
17	Gravel 54% crushed	0 75 to 2	7 6	7 5	6 4	No Test	3,537	4 56	0 278	1 182		7 25	20 8	1 2
Project D—Constants Grout mix, 0–14 sand, $\frac{w}{c}$, flow, compaction by 5 8 ton tandem roller Variables type and size of coarse aggregate, no initial compaction on gravel, twice over on stone and slag, final compaction 4 or 5 times over as required														
18	Limestone	0 5 to 1 5	8 2	6 9	3 6	No Test	3,310	Penetration Failure, Section Removed, see 18R				7 75	21 4	1 2
19	Limestone	1 to 2	8 2	7 1	6 0	404	3,743	4 79	0 309	1 346		7 75	21 4	1 2
20	Slag	0 75 to 2	7 8	7 0	6 1	753	3,620	4 51	0 291	1 245		7 75	21 4	1 2
21	Gravel	0 75 to 2	7 0	6 8	6 1	548	4,013	4 72	0 304	1 118		7 75	21 4	1 2
22	Gravel	0 5 to 1 5	7 2	7 2	6 3	620	3,477	4 34	0 281	1 135		7 75	21 4	1 2
Project E—Constants Grout mix, 0–8 sand, $\frac{w}{c}$, flow, compaction by 5 8 ton tandem roller Variables type of coarse aggregate, no initial compaction on gravel, 8 times over on slag and stone														
23	Limestone	1 to 2	8 0	7 0	5 9	499	4,017	4 67	0 293	1 339		7 5	21 0	1 2
24	Slag	0 75 to 2	8 1	7 0	3 9	No Test	4,330	Penetration Failure, Section Removed, see 24R				7 5	21 0	1 2
25	Gravel	0 75 to 2	7 6	7 4	6 6	740	4,417	4 68	0 293	1 153		7 5	21 0	1 2
Project F—Constants Grout mix, 0–8 sand, $\frac{w}{c}$, flow, no initial compaction Variables type of coarse aggregate, final compaction on stone and slag by 5 8 ton tandem roller, on gravel by hand														
26	Limestone	1 to 2	7 9	7 6	6 1	No Test	4,893	5 64	0 354	1 283		7 5	20 5	1 2
27	Slag	0 75 to 2	7 8	7 3	5 9	432	3,920	4 69	0 293	1 310		7 5	20 5	1 2
28	Gravel	0 75 to 2	7 0	6 3	6 3	452	4,027	4 73	0 295	1 114		7 5	20 5	1 2

TABLE III—*Continued*

Project G—Constants Grout mix, 0–8 sand, $\frac{w}{c}$, flow, coarse aggregate, compaction by 5 8 ton tandem roller, initial 8 times over, final 4 times over Variable subgrade treatments *Section 29* A 2 5 in layer of sand, mulched into subgrade and rolled *Section 30* a 2 1 in layer of limestone screenings, mulched into subgrade and rolled *Section 31* a 1 6 in layer of ½- to ¾-in gravel rolled into subgrade *Section 32* a 1 6 in layer of crusher run limestone (dust to 1½ in) rolled into subgrade *Section 33* one layer (1 4 in) of 1 to 2 in limestone rolled into subgrade *Section 33A* no treatment

Section Number	Coarse Aggregate: Kind	Coarse Aggregate: Size	Coarse Aggregate: Loose Depth, Inches	Coarse Aggregate: Final Depth, Inches	Grouted Depth, Inches	Strength Tests Lb per Sq In: 7 Day Modulus of Rupture	Strength Tests Lb per Sq In: 28 Day Compression	Materials: Cement Sacks per Cu Yd	Materials: Sand Cu Yd per Cu Yd	Materials: Coarse Agg Cu Yd per Cu Yd	Vibratory Equipment	$\frac{w}{c}$ Gallons per Sack	Flow Time, Seconds	Grout Mix
29	Limestone	1 to 2	7 6	6 7	5 5	568	4,103	4 80	0 299	1 360		7 5	20 8	1 2
30	Limestone	1 to 2	7 7	6 8	6 2	540	3,830	4 38	0 276	1 227		7 5	20 8	1 2
31	Limestone	1 to 2	7 9	6 7	6 7	563	3,540	4 32	0 271	1 163		7 5	20 8	1 2
32	Limestone	1 to 2	7 9	6 6	6 5	587	3,670	4 44	0 279	1 200		7 5	20 8	1 2
33	Limestone	1 to 2	7 9	6 7	6 7	536	3,980	4 76	0 298	1 173		7 5	20 8	1 2
33A	Limestone	1 to 2	7 8	6 9	5 9	536	3,903	5 23	0 329	1 312		7 5	20 8	1 2

Project H—Constants Grout mix, 0–8 sand, $\frac{w}{c}$, flow, compaction by 5 8 ton tandem roller Variables type of coarse aggregate, nominal thickness 8 in on sections 34 and 35, 10 in on 36 and 37, no initial compaction on gravel, twice over on stone, final compaction as required

Section Number	Coarse Aggregate: Kind	Coarse Aggregate: Size	Coarse Aggregate: Loose Depth, Inches	Coarse Aggregate: Final Depth, Inches	Grouted Depth, Inches	Strength Tests Lb per Sq In: 7 Day Modulus of Rupture	Strength Tests Lb per Sq In: 28 Day Compression	Materials: Cement Sacks per Cu Yd	Materials: Sand Cu Yd per Cu Yd	Materials: Coarse Agg Cu Yd per Cu Yd	Vibratory Equipment	$\frac{w}{c}$ Gallons per Sack	Flow Time, Seconds	Grout Mix
34	Limestone	1 to 2	10 0	8 8	8 0	567	3,590	4 87	0 304	1 249		7 5	20 8	1 2
35	Gravel	0 75 to 2	9 4	8 5	8 2	529	3,672	4 46	0 280	1 147		7 5	20 8	1 2
36	Limestone	1 to 2	12 0	10 6	9 9	534	3,346	4 53	0 284	1 209		7 5	20 8	1 2
37	Gravel	0 75 to 2	11 8	11 1	10 6	459	3,442	4 57	0 287	1 130		7 5	20 8	1 2

Project J—Constant Grout mix, 0–8 sand, $\frac{w}{c}$, flow, no initial compaction, final compaction by vibratory screed Variable type of coarse aggregate, nominal thickness, 8 in on sections 38 and 39, 10 in on 40 and 41

Section Number	Coarse Aggregate: Kind	Coarse Aggregate: Size	Coarse Aggregate: Loose Depth, Inches	Coarse Aggregate: Final Depth, Inches	Grouted Depth, Inches	Strength Tests Lb per Sq In: 7 Day Modulus of Rupture	Strength Tests Lb per Sq In: 28 Day Compression	Materials: Cement Sacks per Cu Yd	Materials: Sand Cu Yd per Cu Yd	Materials: Coarse Agg Cu Yd per Cu Yd	Vibratory Equipment	$\frac{w}{c}$ Gallons per Sack	Flow Time, Seconds	Grout Mix
38	Limestone	1 to 2	9 3	8 9	8 9	441	3,727	5 51	0 346	1 044		7 5	21 0	1 2
39	Gravel	0 75 to 2	8 7	8.5	8 5	407	3,170	4 69	0 295	1 023		7 5	21 0	1 2
40	Limestone	1 to 2	11 2	10 8	10 8	424	2,987	5 68	0 355	1 033		7 5	21 0	1 2
41	Gravel	0 75 to 2	11 5	10 6	10 6	477	2,512	4 64	0 291	1 090		7 5	21 0	1 2

Project K—Constants Grout mix, 0–8 sand, initial and final compaction by vibratory screed Variables $\frac{w}{c}$ 6 9 gal per sack on section 42, 7 5 gal per sack on other sections, flow, type and size of coarse aggregate														
42	Limestone	1 5 to 3	6 7	6 7	6 3	652	4,808	5 41	0 340	1 062	Screed	7 5	23 5	1 2
43	Limestone	1 to 2	7 0	6 6	6 4	522	4,203	5 58	0 350	1 096	Screed	7 5	21 1	1 2
44	Limestone	0 5 to 1 5	7 4	6 9	2 8	Penetration Failure, Section Removed, see 44R					Screed	7 5	21 1	1 2
45	Slag	0 75 to 2	7 2	6 8	6 6	469	2,643	4 14	0 261	1 094	Screed	7 5	21 1	1 2
46	Gravel	0 75 to 2	7 9	7 2	7 0	582	4,138	4 93	0 307	1 126	Screed	7 5	21 1	1 2
47	Gravel	0 5 to 1 5	7 5	7 3	3 7	Penetration Failure, Section Removed, see 47R					Screed	7 5	21 1	1 2
Project L—Constants Grout mix, 0–8 sand, no initial compaction, final compaction by vibration Variables $\frac{w}{c}$, flow time, type and size of coarse aggregate, vibratory equipment														
48	Limestone	1 5 to 3	6 9	6 9	6 7	550	4,537	5 27	0 328	1 029	Screed	8 0	19 2	1 2
49	Limestone	1 to 2	6 9	6 8	6 7	550	4,410	5 24	0 327	1 024	Screed	7 5	21 0	1 2
50	Limestone	0 5 to 1 5	6 9	6 4	6 3	638	3,657	5 08	0 316	1 104	Screed and Puddler	8 0	19 2	1 2
51	Slag	0 75 to 2	6 5	6 0	6 0	572	3,952	5 16	0 326	1 082	Puddler	7 5	20 4	1 2
52	Gravel	0 75 to 2	7 1	6 7	6 6	544	3,930	4 70	0 297	1 079	Screed	7 5	20 4	1 2
53	Gravel	0 5 to 1 5	6 9	6 6	6 6	551	3,682	4 46	0 277	1 040	Puddler	7 5	20 8	1 2
Project M—Constant Grout mix, 0–8 sand, no initial compaction, final compaction twice over by vibratory puddler Variable $\frac{w}{c}$, flow time, type and size of coarse aggregate														
54	Limestone	1 5 to 3	6 5	6 5	6 4	494	4,782	5 51	0 344	1 021	Puddler	6 5	31 0	1 2
55	Limestone	1 to 2	6 7	6 3	6 2	528	3,915	5 29	0 333	1 085	Puddler	7 0	22 5	1 2
56	Limestone	0 5 to 1 5	6 8	6 2	6 1	548	3,815	4 84	0 301	1 124	Puddler	7 5	20 0	1 2
57	Slag	0 75 to 2	6 9	6 2	6 1	523	4,145	5 13	0 323	1 134	Puddler	7 0	22 8	1 2
58	Gravel	0 75 to 2	6 7	6 2	5 9	528	4,173	4 86	0 306	1 133	Puddler	7 0	25 5	1 2
59	Gravel	0 5 to 1 5	6 8	6 6	6 5	556	3,438	4 15	0 261	1 060	Puddler	7 5	22 0	1 2

TABLE III—*Concluded*

Section Number	Coarse Aggregate: Kind	Coarse Aggregate: Size	Coarse Aggregate: Loose Depth, Inches	Coarse Aggregate: Final Depth, Inches	Grouted Depth, Inches	Strength Tests Lb per Sq In: 7 Day Modulus of Rupture	Strength Tests Lb per Sq In: 28 Day Compression	Materials: Cement Sacks per Cu Yd	Materials: Sand Cu Yd per Cu Yd	Materials: Coarse Agg Cu Yd per Cu Yd	Vibratory Equipment	$\frac{w}{c}$ Gallons per Sack	Flow Time, Seconds	Grout Mix
Project N—Constant coarse aggregate, 0–8 sand, flow, compaction by 5 8 ton tandem roller, twice over initially Variable Grout mix, $\frac{w}{c}$ final compaction 4 to 8 times over as required Size of coarse aggregate subgrade treatment sections 60 and 61, 1 in layer of ¼- to ¾-in gravel rolled in and covered with limestone screenings and rolled Sections 62 and 63, one layer (1 3 in) of 1- to 2-in limestone rolled in and covered with limestone screenings and rolled														
60	Limestone	1 5 to 3	7 1	6 4	5 8	416	3,264	3 91	0 305	1 213		9 0	20 3	1 2 5
61	Limestone	1 to 2	7 4	6 4	6 2	420	3,328	3 78	0 298	1 187		9 0	20 3	1 2 5
62	Limestone	1 5 to 3	6 7	6 0	5 6	441	3,462	3 97	0 342	1 187		9 9	20 3	1 2 75
63	Limestone	1 to 2	7 1	6 2	5 7	416	3,072	3 97	0 343	1 238		9 9	20 3	1 2 75
Project P—0–8 sand, compaction by 5 8 ton tandem roller, initial twice over, final five times over														
64	Limestone	1 to 2	7 8	7 1	6 4	536	4,184	5 32	0 334	1 215		6 75	25 6	1 2
Project Q—Constants Grout mix, 0–8 sand, compaction by 5 8 ton tandem roller, initial twice over, final five times over Loose coarse aggregate not sprinkled Variable $\frac{w}{c}$, flow														
65	Limestone	1 to 2	7 7	6 9	6 4	544	4,348	5 05	0 316	1 194		7 5	20 9	1 2
66	Limestone	1 to 2	7 7	6 9	6 0	530	4,388	5 09	0 319	1 273		8 5	18 6	1 2
Project R—Constant 0–14 sand, size and type of coarse aggregate, compaction by 5 8 ton tandem roller Variable Grout mix, $\frac{w}{c}$ and flow No initial rolling on section 70, twice over on other sections Final rolling 3½ to 5 times over as required														
67	Limestone	0 5 to 1 5	8 0	7 1	6 3	507	3,307	5 13	0 331	1 267		8 75	18 0	1 2
68	Limestone	0 5 to 1 5	7 7	7 0	6 4	515	4,433	6 09	0 294	1 208		7 75	18 0	1 1 5
69	Limestone	0 5 to 1 5	8 0	7 1	6 6	563	4,127	6 93	0 223	1 213		6 75	18 0	1 1
70	Limestone	0 5 to 1 5	7 7	7 3	6 9	509	3,927	5 66	0 365	1 123		7 75	20 0	1 2

Project S—Constant Grout mix, 0–8 sand, $\frac{w}{c}$, flow, compaction by 3 3 ton tandem roller Variables type and size of coarse aggregate, no initial compaction on gravel, twice over on stone Final compaction 2 to 6 times over as required

3R	Gravel	0 75 to 1 5	7 1	7 0	6 7	335	3,644	4 82	0 301	1 055		7 5	21 0	1 2
5R	Limestone	1 5 to 3	6 9	6 8	6 0	536	5,200	5 27	0 330	1 149		7 5	21 0	1 2
6R	Limestone	1 to 2	7 2	6 7	5 6	458	4,582	5 38	0 337	1 285		7 5	21 0	1 2

Project T—Constant Grout mix, 0–8 sand, $\frac{w}{c}$, flow, no initial compaction, final compaction 4 times over with 3 3 ton tandem roller Variable type and size of coarse aggregate

15R	Limestone	1 to 2	7 0	6 8	6 2	404	3,926	5 65	0 354	1 130		7 5	21 0	1 2
18R	Gravel	0 75 to 1 5	7 0	6 8	6 3	412	3,818	4 64	0 290	1·107		7 5	21 0	1 2

Project U—0–8 sand; no initial compaction, final compaction 5 times over with 3 3 ton tandem roller Johnson method finish

24R	Limestone	1 to 2	7 1	7 1	6 4	592	4,942	5 46	0 342	1 102		7 5	21 0	1 2

Project V—Grout mix, 0–8 sand, $\frac{w}{c}$, flow, no initial compaction; final compaction by hand Variable coarse aggregate

44R	Limestone	1 to 2	6 6	6 5	6 3	377	3,967	5 35	0 336	1 041		7 5	21 0	1 2
47R	Gravel	0 75 to 2	6 9	6 6	6 6	335	3,680	4 79	0 300	1 045		7 5	21 0	1 2

Notes

Joints All joints between sections are 1 x 6 in untreated yellow pine boards except as follows

Between 60 and 61 and between 64 and 65 1 in poured type bituminous joint

Between 65 and 66 and between 5R and 6R 1 in premolded type bituminous joint

Between 23 and 24R ½ in premolded type bituminous joint

Thickness All sections of 6 in nominal thickness except as noted

Sprinkling All coarse aggregates sprinkled before grouting except as noted

Proportions All grout mixes by weight

Quantities All quantities are total materials per cu-yd of grouted pavement

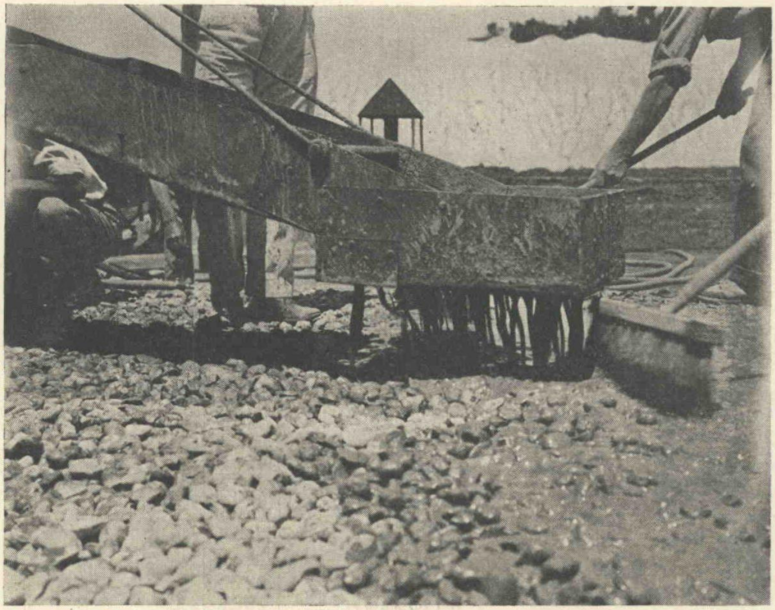

Figure 3. Grout was applied through a special distributing box on the end of the mixer spout.

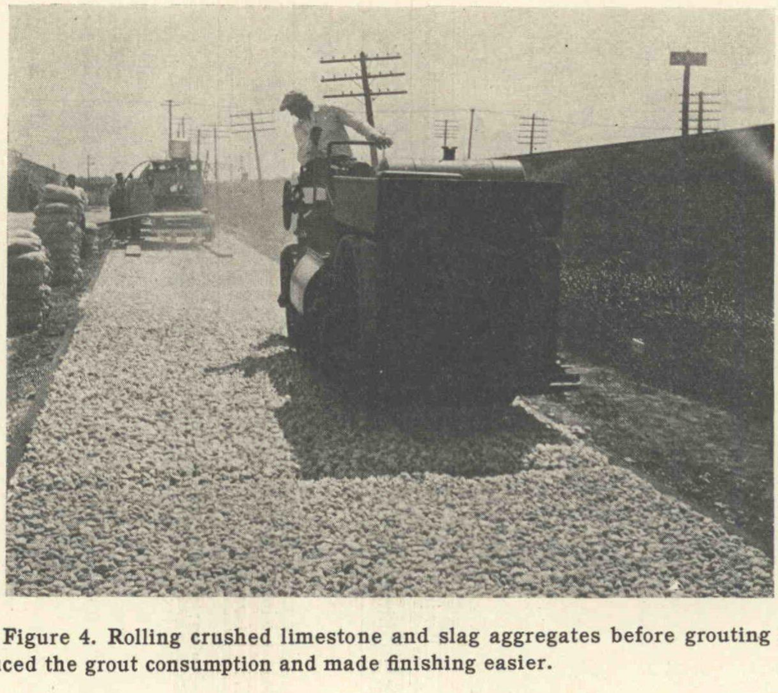

Figure 4. Rolling crushed limestone and slag aggregates before grouting reduced the grout consumption and made finishing easier.

compaction, although subgrade losses are but little lower, than those obtained with the 5.8 ton roller.

Hand tamping gives strengths and compaction less than those obtained with the 3.3 ton roller. The loss of coarse aggregate into the subgrade is negligible.

Rolling crushed stone or slag before grouting reduces voids and effects a considerable saving in grout.

Loss of crushed stone or slag into the subgrade is greater when the coarse aggregate is not rolled before grouting than when it is rolled before and after grouting.

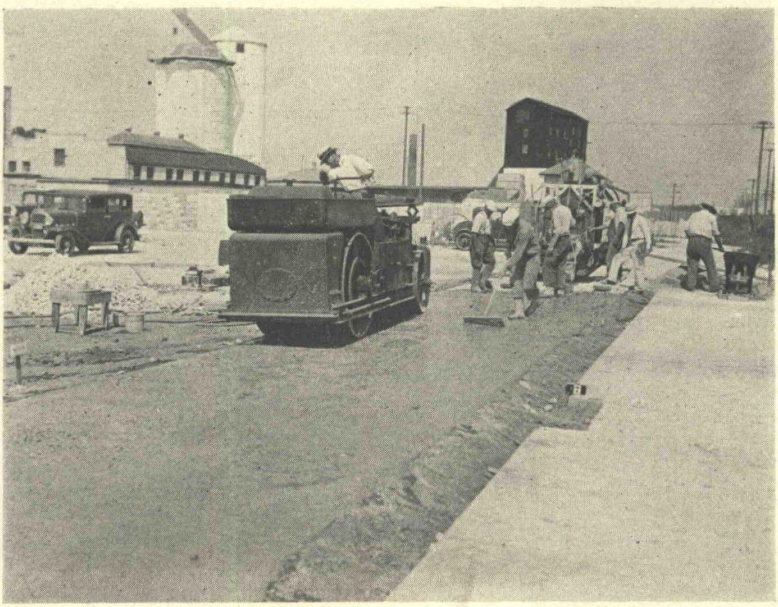

Figure 5. All grouted aggregates could be successfully rolled. For best results rolling should not follow too soon after grouting.

Pertaining to construction methods. Sprinkling coarse aggregate ahead of grouting aids penetration. The addition of one-half gallon of mixing water per sack of cement overcomes the flow resistance of grout caused by the omission of sprinkling.

Data on the readiness with which grout flows through coarse aggregate, obtained from observations in inspection holes dug just ahead of grouting, is a reliable index of penetration. Actual penetration is assured if grout enters *at the bottom* of the inspection holes when the grout on the surface is one foot or more away (Fig. 6).

Drains from the subgrade should be provided if observations in inspection holes dug in the coarse aggregate just ahead of grouting show an excessive amount of free water flowing ahead of the grout.

The proper time interval between grouting and final compaction by mechanical means may be gauged by the time when there is no appreciable amount of free surface water and when the slab is partially stabilized by early hardening of the grout. Wet subgrades, low temperatures and high humidity increase the time interval.

The use of a longitudinal tamping template following final compaction is of material assistance in securing a good riding surface.

Wood, poured or premolded bituminous transverse expansion joints are satisfactory for cement bound macadam.

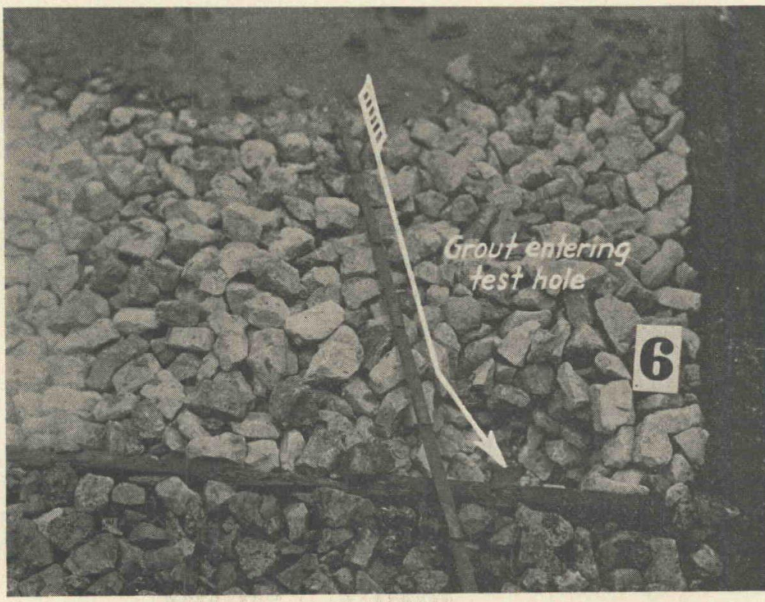

Figure 6. Inspection holes dug in the coarse aggregate ahead of grouting proved to be a reliable index of the completeness of penetration.

Pertaining to strengths. Strengths of cement bound macadams are comparable to strengths obtained with concretes having water-cement ratios the same as those existing in the grout after final compaction.

Pertaining to quantities of materials. The amount of coarse aggregate required depends on amount of compaction and the loss into the subgrade.

The quantity of cement and sand required depends on the proportions of the grout, the type, size and grading of coarse aggregate and the amount of initial compaction.

The average absolute volume of coarse aggregate comprises from 54 to 64 per cent of the total volume of fully grouted pavement.

DISCUSSION
ON
CEMENT MACADAM EXPERIMENTS

MR R R LITEHISER, *Ohio State Highway Department?* What was the approximate length of time after placing of grout that was found best for using the roller?

MR FLEMING That will vary with the amount of water in the grout, and the rapidity with which it is lost by absorption into the subgrade and aggregate, by evaporation from the surface and by spilling over the forms On our test road last summer, with temperatures as high as 90 degrees, rolling after grouting was delayed from 45 minutes to one hour at times Usually the time will range from 30 minutes to one hour but under certain conditions may be much longer

MR LITEHISER When you use the vibrator, is a roller also used?

MR FLEMING. No

MR W A SHELTON, *U S Bureau of Public Roads* I would like to know how the cost would compare with an eleven or nine inch concrete road

MR FLEMING Cement bound macadam will cost on the average from 85 to 90 per cent of mixed concrete per unit volume As coarse aggregate goes down in price, cement bound macadam costs will also drop in comparison with mixed concrete because of the larger volume of coarse aggregate used

GRAVEL TYPE STABILIZED SURFACES FOR SECONDARY ROADS[1]

By Ray B Travers, *County Superintendent of Highways, Syracuse, New York,* and W B Hicks, *Solvay Process Company*

SYNOPSIS

Low cost stabilized gravel type surfaces have been developed as one approach to the problem of improvement of secondary roads The process described in this paper consists in building on a graded road a surface course of a mixture of coarse aggregate, fine aggregate, silt and clay, proportioned for maximum stability according to the latest methods, and treated with a deliquiscent salt to preserve the stability and keep down the dust by maintaining it in a damp condition The report discusses specifications for materials and mixtures and describes the methods used on 300 miles of roads in Onondaga County, New York

There are approximately 2,300,000 miles of unimproved secondary roads in this country, representing 80 per cent of the total road mileage of the United States In recent years there has been an increasing demand for the development of a low cost road construction program that would involve the improvement of these secondary roads One approach to this problem has been the development of the low cost stabilized gravel type roads

Fortunately, prior to this development the Bureau of Public Roads and other research organizations had already made extensive investigations on subgrade and dirt road soils, particularly as regards their stability They had developed methods of testing such soils and had shown that proper mixtures of coarse aggregate, fine aggregate, silt and clay make satisfactory wearing courses for dirt roads when properly maintained in a slightly moist condition During the last two years the results of these investigations have been put on a practical basis through large scale experiments by certain research groups, through many test projects, and through road construction mileage in various localities

Grading and draining are the first steps in this development, and are required alike for all improved roads These items usually absorb a large share of the cost in the construction of a stabilized gravel type road

Preparation of the wearing course is the second step and usually absorbs the second largest share of the cost This involves adding cer-

[1] This report was received too late for inclusion on the program of the Twelfth Annual Meeting, but was suggested for publication by the Committee on Design as information on the development of one interesting type of low cost road No conclusions or recommendations are offered by the Committee Observations of stabilized surfaces, built as described in this report, should be continued in order to determine their behavior under different weather conditions

tain soil or aggregate ingredients to the subgrade to stabilize the surface or else adding stabilized soil mixtures of suitable thickness Selection of proper materials for this purpose is of paramount importance

Treatment with a deliquescent salt is a third and important step, but is a minor item as regards cost

Maintenance is an important consideration because stabilized gravel type roads must be properly maintained to keep them in satisfactory condition

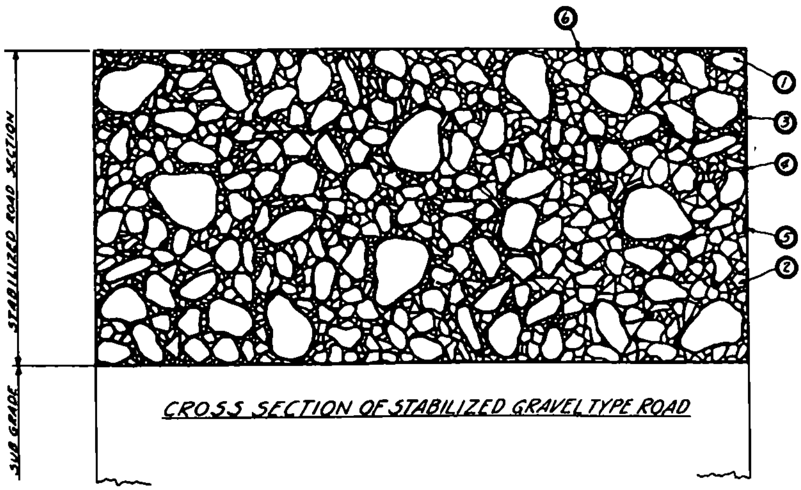

Figure 1

1. Gravel aggregate. Screen size—on No. 10 sieve—through ¾-inch +, stone larger than ¾-inch should be raked out of the top surface, to facilitate blading This ingredient contributes rigidity and high internal friction and adds to the mechanical stability of the road surface

2. Coarse sand. Screen size—through No. 10 sieve—over No 40 sieve Serves same purpose as gravel, helps lock gravel in place

3 Fine sand Screen size—through No 40 sieve—over No 270 sieve. Serves same purpose as gravel, fills voids in coarse sand, non-expansive ingredient

4. Silt. Has high capillary properties, serves as a reservoir for calcium, contributes but little to rigidity of road surface, fills voids in fine sand, expands when moistened Has no cohesive properties or toughness.

5 Clay Supplies cohesion or toughness, has expansive properties, fills voids completely making road surface impermeable, serves as a reservoir for calcium.

6. Calcium Tends to keep moisture of road surface at an optimum content which maintains toughness of road. It is driven into the road surface by rains, reappears at the surface by capillarity in dry weather. Reduces evaporation and attracts moisture from the atmosphere

The ideally stabilized gravel type wearing course may be defined as one that will give proper support to wheel loads, that will not become slippery and rut up in wet weather, and will not become dusty in dry weather In general the aggregate should be embedded in the soil mortar, leaving the surface smooth with a mosaic appearance and practically free from floating material An idealized cross section of such a wearing course is shown in Figure 1

The road to be improved is first brought to subgrade and provided with adequate culverts and side ditches A wearing course of stabilized material of suitable thickness (minimum three inches) is then built onto the subgrade A typical section of such a road as constructed in Onondaga County, N Y , is shown in Figure 2

COMPOSITION AND CHARACTERISTICS OF THE WEARING COURSE

Materials specifications for the wearing course have been proposed by the Calcium Chloride Association as follows.

(1) *Coarse aggregate* comprises the portion retained on a No 10 sieve and may consist of gravel, crushed stone, or slag, or combinations thereof In all cases 90 per cent chall pass a sieve having ¾ inch square openings, and 100 per cent shall pass a sieve having 1 inch square openings

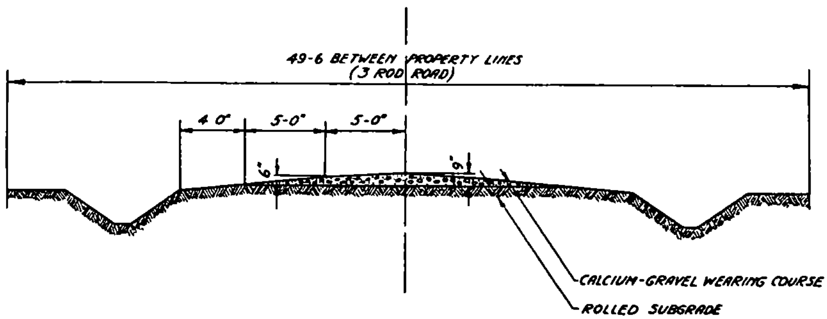

Figure 2 Typical Section of Stabilized Gravel Type Road

(2) *Fine aggregate* comprises the portion passing a No 10 sieve and retained on a No 270 sieve, and may consist of sand, crushed stone, or slag, or combinations thereof

(3) *Natural soil binder* shall consist primarily of fine soil particles and shall contain no stones which will not conform to the course aggregate requirements, and also shall not contain more than 5 per cent organic matter

Its binding properties shall be such that it provides the required physical properties to the finished mixture as hereafter described

(4) *Finished mixture* shall consist of coarse aggregate, fine aggregate, and natural soil binder, supplied individually or in combination, to give the following proportions:

Passing 1 inch sieve*	100 per cent
Passing ¾ inch sieve	95 per cent
Passing No 4 sieve	50 to 85 per cent

Passing No 10 sieve	40 to 65 per cent
Passing No 40 sieve	25 to 50 per cent
Passing No 270 sieve†	10 to 25 per cent

The fraction passing the No 270 sieve shall not be more than two-thirds of the fraction passing the No 40 sieve

The fraction passing the No 40 sieve shall have a plasticity index between 6 and 14 as determined by physical test methods of the U S Bureau of Public Roads (Described in "Public Roads," Vol 12, No 8, October, 1931)

* Sieves for gradation analyses shall have square openings and shall be the U S Standard Series for No 4 and finer sieves

† Standard settling rate test ordinarily used in the mechanical analysis eliminates the need of the No 270 sieve

(5) *Calcium chloride* shall conform to the requirements of the American Society for Testing Materials, Tentative Specifications for Calcium Chloride for Dust Prevention (D98-30T)

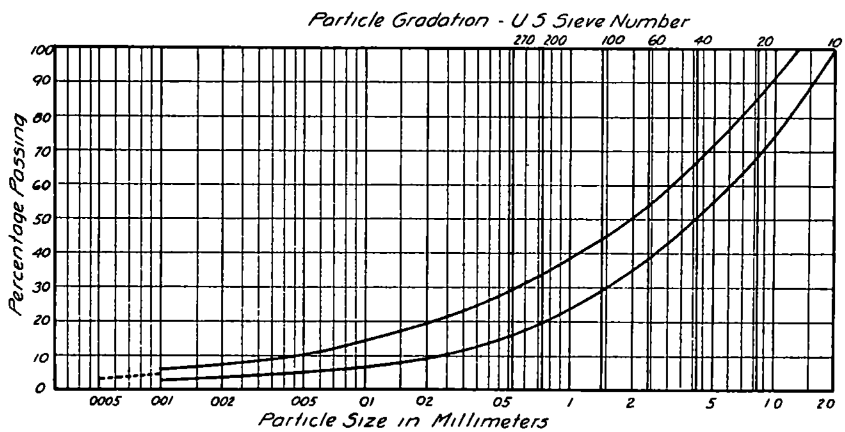

Figure 3. Grading of Stable Soil Mortars

(6) Quantities of pit run gravel, stone, slag, soil binder, etc , required to produce the finished mixture described above shall be determined by laboratory tests on the supplies of materials which are available and shall be specified in cubic yards or tons per mile of road

"Figure 3 shows a chart representing grading of good soil mortars, redrawn from one published by the Bureau of Public Roads in "Public Roads," Vol 12, No 5, July 1931 This chart includes only the soil mortar or material passing the No 10 sieve To use the chart, a mechanical or sieve analysis of the soil is first made, and the fraction passing the No. 10 sieve calculated to 100 per cent The resulting percentages passing the several sieves are plotted on the chart against sieve openings or particle sizes. If the curve so obtained falls within the shaded area, the gradation will be satisfactory. When such a soil mortar is mixed with 35 to 60 per cent of graded coarse aggregate, the mixture should have the proper gradation for a satisfactory wearing course

"Figure 4 is a similar chart but representing the entire wearing course including both coarse aggregate and soil mortar To use this chart, a mechanical or sieve analysis is first made, and the fraction passing a 1-in sieve is calculated to 100 per cent The resulting percentages passing the several sieves are plotted on the chart against sieve openings or particle sizes If the curve so obtained falls within the shaded area and has a distinct continuous slope, the material may be considered to have the proper gradation for a satisfactory wearing course "

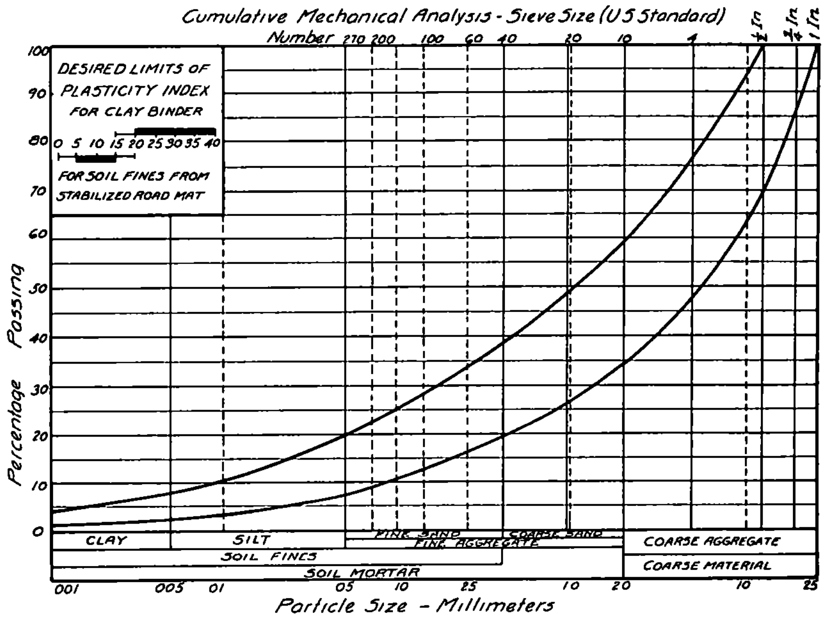

Figure 4 The data of Figure 3 Revised to Include 40 to 55 per cent of Coarse Material Retained on the No 10 Sieve

Analytical data relative to the wearing courses on several stabilized gravel type roads in Onondaga County, N Y , are given in Tables I, II and III The data show that the Berwin Road and Kieffer gravel fall within the limits of the Bureau of Public Roads chart (Figure 3) All of the other wearing courses fall within the limits of this chart for the amount passing the No 40 sieve, but fall above the limits for the amount passing the No 270 sieve They all meet the specifications given herein except the Lakeland Road The latter wearing course is slightly low in the amount passing the No 40 sieve and also in the plasticity index, and the fraction passing the No 270 sieve is more than two-thirds of that passing the No 40 sieve

A few comparatively simple tests will determine whether or not a given soil mixture will remain stable in the wearing course under traffic, and what materials if any need to be added to the subgrade to stabilize it They involve gradation analysis and the determination of the plasticity index Methods of making these tests have been developed by the Bureau of Public Roads ("Public Roads," Volume 12, No 8, October, 1931) After tests have been completed on materials to be used in constructing the wearing course, the quantity of the several materials required to be mixed together to form a product which will meet the specifications and which will conform to the limits given in the chart (Figure 4) should be calculated

Occasionally run of bank gravel will contain about the right proportions of coarse aggregate, fine aggregate, silt and clay to form a stabilized road surface

TABLE I

ANALYSIS OF WEARING COURSES ON GRAVEL TYPE ROADS IN ONONDAGA COUNTY, N Y*

	Berwin Road	7th North Street	Lakeland Road	Kieffer Gravel†
	Per Cent	*Per Cent*	*Per Cent*	*Per Cent*
Coarse Aggregate (Between 1 inch and No 10 sieve)	49 0	36 3	61 1	47 3
Coarse Sand (Between No 10 and No. 40 sieves)	19 8	24 6	15 3	24 6
Fine Sand (Between No 40 and No 270 sieves)	17 1	15 6	6 9	17 6
Silt and Clay (Finer than No 270 sieve)	14 1	23 2	16 7	10 5
Plasticity index	8	7	5	7

* Stones larger than 1 inch not included in sample

† Used in construction of wearing course on a good many roads

CONSTRUCTION PRACTICE IN ONONDAGA COUNTY, NEW YORK

The roadway to be improved is first brought to grade and provided with adequate culverts and ditches It is then thoroughly compacted with a roller weighing not less than ten tons or rammed in places that cannot be rolled Soft spongy ground that will not compact under the roller and silt areas that may transmit large volumes of water are removed, the excavation filled with suitable material and then thoroughly rolled or rammed

Where the ground water is near the original surface, the subgrade is built up or the side ditches are made deep enough to depress the ground water level three or four feet below the crown of the road Ground water level means the depth at which water will stand in a well or pit

Material from the side ditches is generally used to build up the subgrade provided it will stand the weight of the ten ton roller

In case silty or other soils are encountered which retain an excessive amount of water and are subject to swelling and heaving under the action of frost, and yet are not considered detrimental enough to require replacement, tile or other means of subdrainage are provided

After the grading and draining have been completed, a wearing course of suitable thickness is built onto the subgrade This wearing course consists of a mixture of coarse aggregate, fine aggregate, silt and clay, as described in the specifications quoted and as illustrated in Figure 4

The wearing course may be built up in two different ways In one method selected materials are added to and mixed with the subgrade soil in the proper proportions to make a stabilized mixture of suitable

TABLE II

GRADATION ANALYSIS OF MATERIALS PASSING NO 10 SIEVE FROM WEARING COURSES IN ONONDAGA COUNTY, NEW YORK

	Limits of Chart Figure 3	Berwin Road	7th North Street	Lakeland Road	Kieffer Gravel
	Per Cent	*Per Cent*	*Per Cent*	*Per Cent*	*Per Cent*
Passing No 10 sieve	100	100	100	100	100
Passing No 40 sieve	50–67	58	61 2	60 7	53 3
Passing No 270 sieve	16–30	27 7	36 6	43 0	20 0

TABLE III

GRADATION ANALYSIS OF MATERIAL PASSING 1 INCH SIEVE FROM WEARING COURSES IN ONONDAGA COUNTY, NEW YORK

	Limits of Specifications	Berwin Road	7th North Street	Lakeland Road	Kieffer Gravel*
	Per Cent	*Per Cent*	*Per Cent*	*Per Cent*	*Per Cent*
Passing 1 Inch sieve	100	100	100	100	100
Passing No 10 sieve	40–65	51 0	63 4	38 9	52 7
Passing No 40 sieve	25–50	31 2	38 8	23 6	28 1
Passing No 270 sieve	10–25	14 1	23 2	16 7	10 5

* Used in construction of wearing course on a good many roads

thickness Thus a part of the subgrade soil is utilized in building up the wearing course The materials to be used are selected and the proportions determined in the manner previously outlined In building up the wearing course, the subgrade surface is first scarified to the desired depth and bladed into windrows at the shoulders. Selected materials are then placed on the roadway in the correct proportions, and thoroughly mixed with the subgrade soil by blading and harrowing

A second method of building up the wearing course is to spread on the subgrade selected materials of the proper composition and physical properties to form a stabilized layer of suitable thickness In this

Figure 5. Road before Start of Construction

Figure 6. Road Under Construction

Figure 7. Typical View Showing Road Under Construction

Figure 8. Typical View Showing Road Under Construction

TABLE IV

AMOUNT OF CALCIUM CHLORIDE REQUIRED FOR MAINTAINING ONE MILE OF ROAD

Width in Feet	Square Yards per Lineal Foot	Square Yards Per Mile	Amount per Square Yard Pounds	Drums 400 Lbs Each	Bags 100 Lbs Each	Tons
			0 5	6	24	1 17
			1 0	12	47	2 34
8	9	4693	1 5	18	71	3 51
			2 0	24	94	4 68
			2 5	30	118	5 85
			0 5	7	30	1 47
			1 0	15	59	2 93
10	1 1	5866	1 5	22	88	4 40
			2 0	30	118	5 86
			2 5	37	147	7 33
			0 5	9	36	1 76
			1 0	18	71	3 52
12	1 3	7040	1 5	27	106	5 28
			2 0	35	141	7 04
			2 5	44	176	8 80
			0 5	10	41	2 06
			1 0	21	82	4 11
14	1 5	8213	1 5	31	124	6 17
			2 0	41	165	8 22
			2 5	51	206	10 28
			0 5	12	47	2 35
			1 0	23	94	4 69
16	1 7	9387	1 5	35	141	7 04
			2 0	47	188	9 38
			2 5	59	235	11 73
			0 5	13	53	2 62
			1 0	27	106	5 23
18	2 0	10560	1 5	39	159	7 85
			2 0	53	212	10 46
			2 5	66	264	13 08
			0 5	15	59	2 94
			1 0	29	118	5 87
20	2 2	11733	1 5	44	176	8 81
			2 0	59	235	11 74
			2 5	74	294	14 68
			0 5	17	65	3 23
			1 0	32	129	6 45
22	2 4	12906	1 5	49	194	9 68
			2 0	65	259	12 90
			2 5	81	323	16 13
			0 5	18	71	3 52
			1 0	35	141	7 04
24	2 6	14080	1 5	53	212	10 56
			2 0	70	282	14 08
			2 5	88	352	17 60

method no part of the subgrade soil is utilized in building up the wearing course. Frequently run of bank gravel is satisfactory for this purpose. In other cases, mixtures of materials from two or more sources may be necessary. In the latter case the materials after having been spread on the road surface must be thoroughly mixed by blading and harrowing.

After the stabilized surface layer has been built on the subgrade, the road is shaped up to the dimensions shown in Figure 2, leaving a slope from crown to shoulder. It is then compacted with the ten ton roller, rolling from the outside toward the center of the road. Any low places are brought to grade by blading, followed by rolling.

In Onondaga County, New York, where some 300 miles of this type of road have been constructed the wearing courses have been built generally from run of bank gravel and the materials selected by the engineer without the aid of a soil analyst. The wearing courses on several roads however, have been made by combining two or more materials, and in some cases soil analyses have aided in the selection of materials. Good wearing courses have been produced by a top dressing of shale sand over a layer of run of crusher limestone and also over sandy gravel. In other cases where the surface contained an excessive amount of clay, slipperiness has been prevented by a top dressing of limestone screenings ($\frac{3}{8}$ inch down).

Hand labor has been used to a large extent in Onondaga County. Most of the grading and drainage work and the spreading of the materials for the wearing course have been done by welfare workers. Figure 5 shows a typical road before construction was started. Figures 6, 7 and 8 show typical views of roads under construction, illustrating hand labor operations. Figure 9 shows the application of calcium chloride, and figures 10, 11, 12 and 13 show typical views of the completed road under traffic.

After the roadway has been provided with a wearing course of material having the proper gradation and physical properties, the stability of this wearing course is developed and preserved by treatment with calcium chloride. For this purpose flake calcium chloride should be sown in place with a calcium spreader. This is best done when the road surface is in a damp condition. In dry sections of the country it may be desirable to sprinkle the road in order to render the wearing course moist before applying the calcium chloride. When the road is constructed in the spring or early summer, the amount of calcium chloride required for the first season will be about two pounds per square yard. This should be applied in two or more applications. The initial application may vary from 0.6 to 1.5 pounds per square yard, followed by lighter applications as required. When the road is constructed in the late summer or fall, the quantity used the first season may be advantageously reduced to 1.5 or 1 pound per square yard.

The quantity of calcium chloride required for treating one mile of road is shown in Table IV.

Figure 9. Application of Calcium Chloride

Figure 10. Completed Road

Figure 11. Completed Road

Figure 12. Completed Road

The method of applying calcium chloride to a road surface is generally well known, and is illustrated in Figure 9.

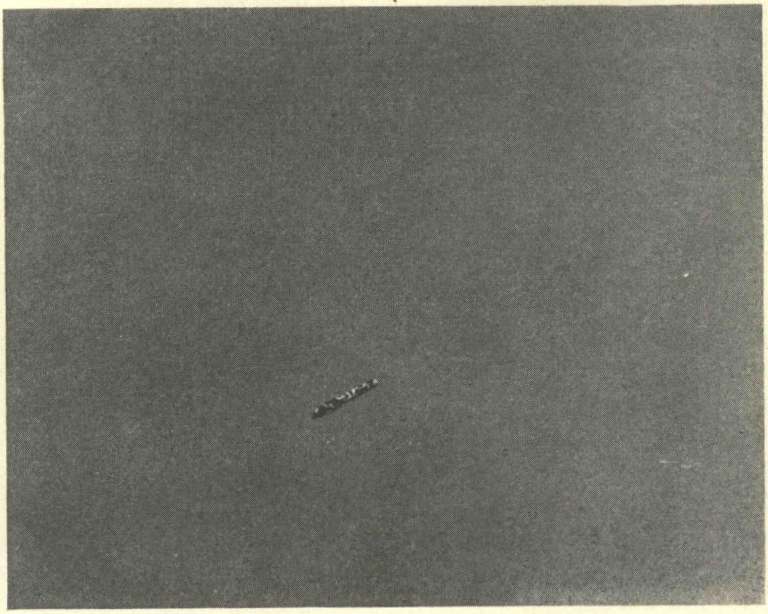

Figure 13. Close up View of Completed Surface

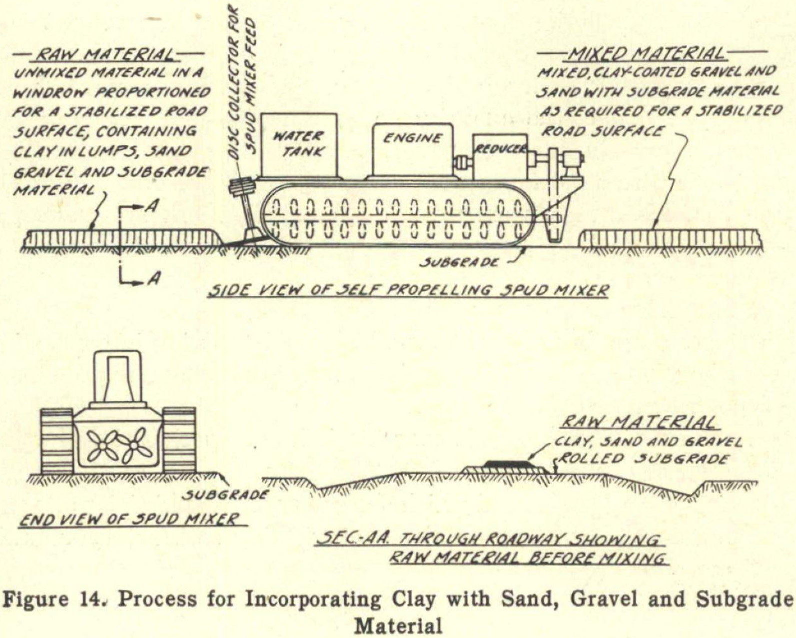

Figure 14. Process for Incorporating Clay with Sand, Gravel and Subgrade Material

The function of the calcium chloride is to retain as much moisture as possible in the surface through its properties of retarding evaporation from the surface and of attracting moisture from the air It is driven into the road surface by rains but is redrawn to the surface in dry weather by capillarity Other things being equal it serves its purpose best when the wearing course has the optimum composition and physical properties In such a case a minimum of calcium will be required With poor gradation and low plasticity index more than the normal amount may be required to keep the proper amount of moisture in the wearing course

After the initial application of calcium chloride has been made, the construction may be considered complete and the road opened to traffic Figures 10, 11, 12 and 13 show typical views of completed stabilized gravel type roads in Onondaga County, New York, after traffic had been on them for some time

MAINTENANCE

Whenever the roadway surface loses its crown and smoothness, it is reshaped by blading from the edges inward in a manner that will gather the loose material from the shoulders, cut off the high places and fill in the low ones This is best done following heavy rains, using light equipment For early spring maintenance heavy equipment may be needed Maintenance material consisting of properly graded soil is added as required

Flake calcium chloride is applied as needed to maintain a moist condition of the surface, thus eliminating dust and preventing raveling of the road surface Under ordinary climatic conditions and normal country road traffic, the quantity required varies from about one to two pounds per square yard per season This is spread on the damp road surface with a calcium spreader The calcium chloride is utilized to the best advantage when several light applications are made per season

RESEARCH SUGGESTIONS

There are a number of problems which need studying in connection with the development of this type of road Among the more prominent problems, we suggest the following:

1 The influence of subgrade characteristics on specifications for the wearing course
2 Utilization of shale sand which shows very little plasticity on tests but which weathers on the road yielding satisfactory binding material
3 Development of machinery for mixing materials forming the wearing course A suggestion along this line is shown in Figure 14.
4. Study of the comparative economics of automobile operation on this type of road

5 Quick testing methods applicable for use in the field
6 Study of maintenance costs

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HIGHWAY LIGHTING RESEARCH

By Arthur F Loewe

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SYNOPSIS

The development of the use of sodium vapor as an illuminant and its application to street and highway lighting is discussed The use of this new source of light will mean many changes in present practices and the research engineers are working on the problem of its most efficient use The sodium lamp derives light from the passage of an electric current through a vapor of sodium with a small quantity of Neon gas When the current is turned on the lamp first glows red as the Neon gas carries the arc and then turns gradually to a golden yellow as the sodium vaporizes The process is fully described The application of the lamp to highway lighting is then considered from the standpoints of "On what does the effective seeing of an object depend?" "How must the sodium vapor illumination be applied?" "What are its visual advantages?" The relation of various kinds of road surfaces to lighting problems is brought out, and the possible future economies that may be realized are discussed Two demonstrations in New Jersey as in accord with the Illuminating Engineering Society's Code are described and illustrated

Even during these past years of economic stress when the expenditure of dollars and even of cents has been very carefully balanced by the value received, there has been in many parts of the country a growing appreciation of the value of adequate street and highway illumination as a safety and saving factor in the affairs of the general public Recently has come the announcement that sodium vapor as an illuminant has been developed to such a point that actual demonstrations may be made in order not only to study its practical application, but also to study its relative desirabilities and limitations as compared to our present modern illuminants

For many years the Research Laboratories have labored with the problem of developing more efficient light sources Hand in hand with this problem have gone the allied problems of engineering development, economic analysis, application studies and the visual reactions of the "human seeing machine." Unfortunately, the acceptance of adequate illumination as well as the acceptance of many other developments having to do with human welfare has lagged materially behind the developments worked out by the research and application engineers

We are indeed standing upon the very threshold of a new and better era in illumination of streets and highways for we now may combine not only the possibilities of potential application and generative efficiencies, but we now also have the realization of the economic advantages of good illumination as well as the fact that we are but "human seeing machines" and react according to the facilities which are provided

In order to develop a sound structure upon which we may build our future store of knowledge and appreciation of sodium vapor as a highway illuminant, the subject should be divided into six parts

1 The scientists' developments applied to present conditions
2 How is sodium vapor used as an illuminant?
3 Why does sodium vapor radiate light?
4 Upon what does the effective seeing of an object depend?
5 How must sodium vapor be applied and what are its visual advantages?
6 What economies may the future hold for the ultimate consumer?

SCIENTISTS' DEVELOPMENTS APPLIED TO PRESENT CONDITIONS

The maximum efficiencies and advantages of many scientific developments are not made immediately available to the general public, if these developments depart radically from used and accepted equipment There has always been a general resistance of the public to adopt quickly major changes If the new developments necessitate entirely new equipment, there are major economic considerations which must be properly evaluated For example, in the automotive field, the lowered wind resistance of the "tear-drop" design has been known This year, however, is the first that we have seen any commercial companies even approximating this shape In the field of illumination, we are all familiar with the slow change that has taken place in the candelabra type of luminaire and the acceptance of adequate and proper illumination from indirect and localized sources

Manufacturers do the great proportion of research and development Their activities are dictated to a considerable extent by commercial expediency Revolutionary developments are very slowly applied to public benefit, on account of the cost and public inertia

The research laboratory developments of high efficiency light generation through sodium vapor must be tempered by the fact that this illuminant, revolutionary as it is, is but a step forward from that of tungsten sources Therefore, the engineer must give due consideration in his developments to the existing 100 volt range multiple systems and 5 to 7 ampere range series systems as are generally found here in the United States He must compete with the simplicity of auxiliary operating devices of the incandescent system as well as its instantaneous starting characteristics and stability of intensity In addition, in order to use efficiently the sodium vapor lamp of low intrinsic brilliancy and large physical dimensions, for highway lighting, he must combat the antiquated though extensively used systems of low mounted pole top units with the light source placed at one side of the road (see Figures 1 and 2) These are all conditions which militate against the most efficient application of the high efficiencies developed in the laboratory lamps.

HOW IS SODIUM VAPOR USED AS AN ILLUMINANT?

The sodium lamp derives light from the passage of an electric current through a vapor of sodium rather than from a tungsten wire; therefore,

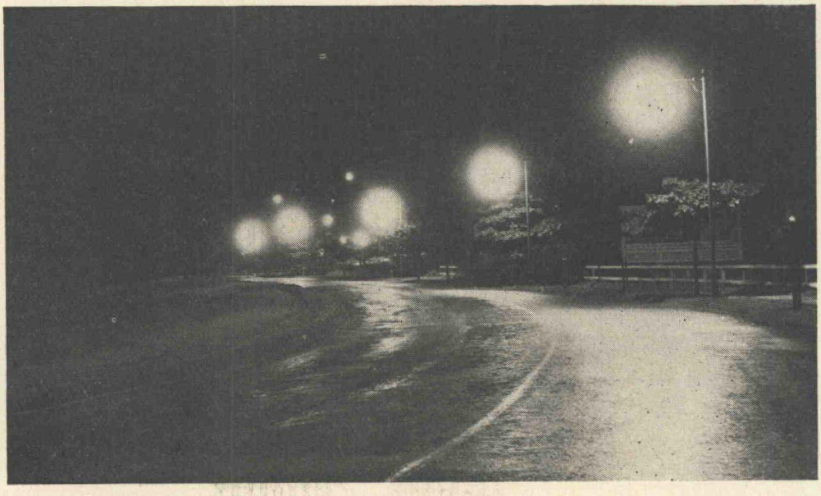

Figure 1. Efficient Light Application 6000 Lumen Lamps—20-Ft. Mounting Height—151-Ft. Linear Spacing. 6-Ft. Bracket. Asphalt Pavement. Note how the specular reflection practically covers the entire traveled roadway.

Figure 2. Inefficient Light Application. 10,000-Lumen lamps—13-Ft. Mounting Height. 173-Ft. Linear Spacing. Lamps 3 ft. back of road surface. Asphalt Pavement. Note how specular reflection does not cover traveled road surface. Lighting conditions would be even worse than indicated on a straight-away.

as with all gaseous conduction lamps, it has the fundamental characteristics of an arc lamp resulting in the need of devices for limiting the rise of current Generally a system of a cathode and two anodes or two cathodes and two anodes is employed in an evacuated bulb (see Figure 3) A carefully measured quantity of pure sodium and a small

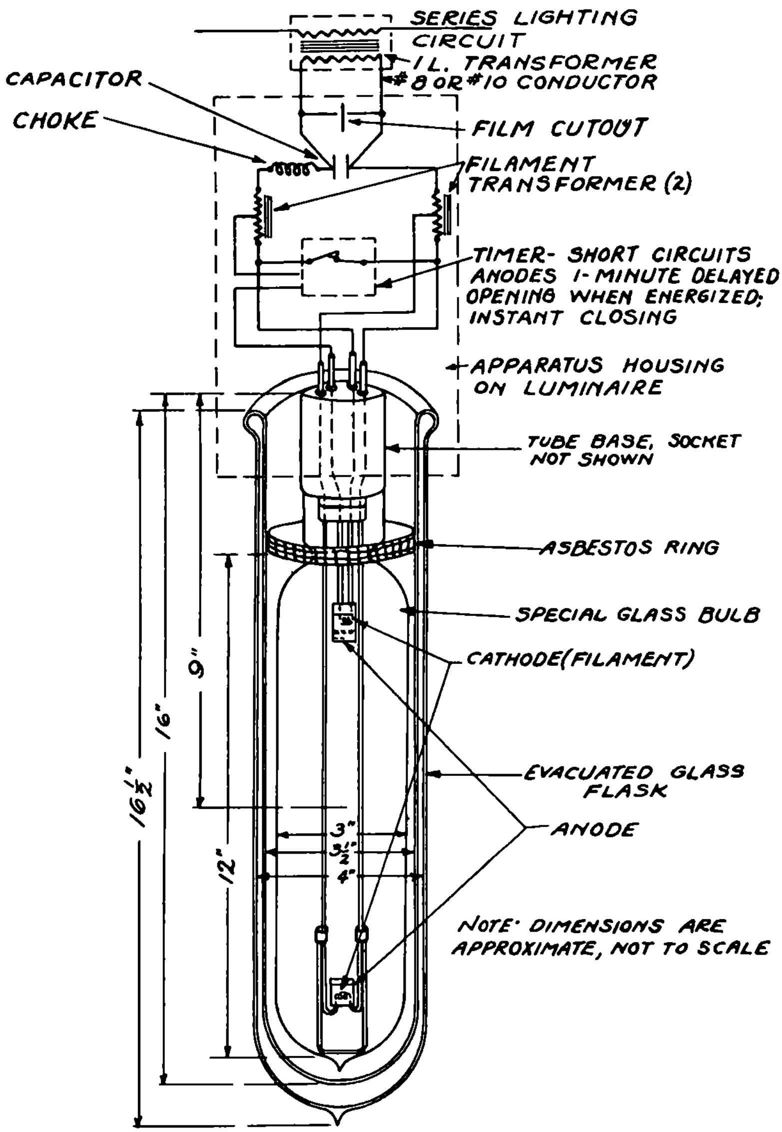

Figure 3. Schematic Diagram, 10,000 Lumens A.C. Sodium Lamp. With Series Transformer.

quantity of Neon gas are placed in the globe which is of special sodium resisting glass When the cathode is heated by a low voltage external circuit and a potential applied between the anodes and cathodes, the lamp glows first red, as the Neon gas carries the arc, turning gradually to a golden yellow as the sodium becomes vaporized in sufficient quantity In this latter state the light is essentially monochromatic, falling close to that portion of the spectrum which provides the greatest retinal sensitivity It is believed that this quality of radiation will remain practically as we have it today, due to the inherent sodium vapor characteristics However, it is anticipated that the equipment used for generation and distribution of light will be further simplified even in the light of the great simplification that has already been realized to date

THE PRODUCTION OF LIGHT FROM A GAS

To obtain a clear yet simple picture of this most interesting phenomenon we must presuppose a particular set of conditions in order to arrive quickly at the critical point of emission of radiant energy recognized commonly as light In the most simple form of sodium vapor tubes, we have the evacuated glass enclosure with a low pressure Neon gas content, plus a definite amount of metallic sodium At one end of the tube is a simple anode and at the other a cathode consisting of materials which freely produce electrons when the cathode is heated through application of voltage across its terminals by a separate source In order to obtain sufficient sodium vapor in the tube, the Neon gas arc is struck by application of proper voltages between anode and cathode, producing sufficient heat to vaporize the sodium When this point is reached the sodium arc strikes and the voltage between the anode and cathode is reduced to the operating level This voltage level must be sufficient to provide accelerating voltage for complete ionization and yet low enough to prevent the destruction of the cathode by bombardment of the positively charged sodium ions We then find a condition of free electrons being released from the heated cathode attempting to travel to the anode through a space (plazma) partially filled with positively charged sodium vapor atoms in equilibrium with their negatively charged electrons traveling in their accustomed orbits

The production of light from a gas is brought about by its bombardment with electrons emitted from the hot cathode When the velocity of the colliding electron is less than a certain value, critical for each gas, the electron simply bounces off suffering a change in direction but causing no disturbance in the atom which it strikes If the velocity of the electron exceeds the critical value it may give up a definite amount of its energy to the atom An atom with this extra amount of energy is said to be excited When this extra energy is given up on the return of the atom to its normal state, the energy is given up as radiation, the wavelength of which depends upon the magnitude of the energy transfer at the time of collision

If the energy of the impinging electron is still higher than in the case just described other excited states may be produced The energy radiated as the atom returns to normal through each of these stages is characteristic of the state in which the transfer is taking place This accounts for the finding of more than the D lines upon spectral analysis

When the energy of bombarding electrons is in excess of another critical value called the ionization potential, the atom may be completely disrupted into a positively charged nucleus or positive ion and a second electron

Electrons on passing through a gas may then do any of three things

1 Suffer an elastic collision with no appreciable loss of energy
2 Excite gas atoms which give up their energy later in the form of radiation
3 Ionize gas atoms producing positive ions, thus neutralizing the electron space charge and rendering the space more conductive.

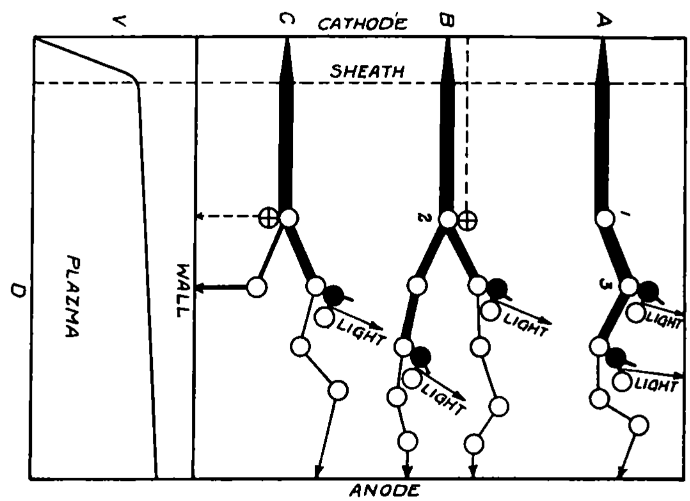

Figure 4. Schematic Diagram Illustrating Production of Light from a Gas

In the usual gas discharge the applied potential is concentrated within a very short distance of the cathode leaving the remainder of the space practically field free The distribution of potential between electrodes is shown at the bottom of Figure 4 The region of high voltage drop is known as the cathode sheath and the remaining region which, as mentioned above, is practically field free, is called the plazma The electrons from the hot cathode are accelerated by the voltage drop in the cathode sheath and then travel through the plazma with practically no further energy increase In other words, the potential energy of the supply voltage is converted into kinetic energy of electrons in the cathode sheath As the electrons diffuse across the plazma this kinetic energy is dissipated in the production of excitation and ionization of the gas atoms The energy consumed by excitation is later given up as radiation while the positive ions and the slow speed electrons (the products of ionization) diffuse to the walls where they recombine and give up their energy in the form of heat

When the sodium lamp is first started the discharge is wholly Neon and the processes taking place are those described above The heat resulting from this arc raises the bulb temperature and vaporizes sodium When this occurs the electrons in the plazma also collide at times with the sodium atoms causing excitation of the latter which in turn produces light

Sodium has the unique characteristic that most of the visible radiation is resonance radiation That is, most of its visible radiation results from the return of its excited atoms to normal from the lowest energy level The wavelength of this radiation is close to the region of maximum retinal sensitivity These two facts, high percentage of resonance radiation, and the high retinal sensitivity to that wavelength, make sodium light production at very high efficiencies possible

Fortunately about 75 per cent of the visible energy is radiated in the yellow or D bands of the spectrum The reason that this is fortunate can only be explained by a discussion of vision

In Figure 4 the energy of the electrons is shown diagrammatically by the width of the line representing its path The electron at A collides elastically with a gas atom of 1 and bounces off in a new direction Its collision with an air atom at 3 results in excitation of the atom and a reduction in the energy of the electron After a second excitation its energy is below the minimum excitation potential and it has only elastic collisions throughout the remainder of its path The excited atom at 3 returns to its normal state and emits a quantum of light

The remaining two electrons at B and C produce excited atoms and also ions The ion produced by the electron (B) moves back to the cathode counteracting electron space charge while that found by (C) together with an electron is shown diffusing to the walls The recombination of ions and electrons at the walls is one of the sources of heat in the arc discharge This production of heat is one of the barriers in practice to obtaining laboratory efficiencies near theoretical maximums

ON WHAT DOES THE EFFECTIVE SEEING OF AN OBJECT DEPEND?

Doctor M Luckiesh of the Lighting Research Laboratories of the General Electric Company at Nela Park in his "The Applied Science of Seeing" has classified the major factors on which visibility of an object depends into eight parts

1 Its size or usually the size of certain critical details In a printed letter, for example, the size of critical details is about one-fifth of the overall size of the letter
2 Its distance from the eyes This datum combined with the physical size of the object is usually expressed as minutes of visual angle

3 Its contrast with its background
4 Its brightness or that of its background which depends upon the reflection factors and the intensity of illumination
5 The time available for seeing
6 The ability of the eyes which depends upon their freedom from defects or the correctness of glasses
7 The ability of the "human seeing machine" which depends upon such factors as intelligence, experience, reaction, concentration and application
8 Various other visual and lighting factors such as glare, adaptation, and the color, brightness and pattern of surroundings

In discussing the relative merits of sodium vapor as an illuminant, the item of contrast of an object with its background, as well as the object's brightness or that of its background, and the element of glare, are those which may have most weight in deciding the relative advantages of one illuminant over another

Increased visual acuity has undoubtedly received more attention from the press than any other factor associated with this new illuminant However, due to the particular monochromatic character of sodium radiation, it has been generally conceded that the big field for this new illuminant is for highway lighting and perhaps some fields of inspection where color discrimination is not a factor In highway lighting, however, visual acuity is not the great factor because visual acuity is generally defined as the ability to distinguish fine details This is an essential of good lighting and good vision, but undoubtedly has been overemphasized in the discussion of seeing When discussed in conjunction with highway lighting, its exact importance is not definitely known, and at present is considered not of greatest moment It may be conceded that visual acuity under sodium illumination is of greater advantage under low levels of brightness than high levels The levels to date, however, that have been investigated are still higher than those levels we are accustomed to dealing with in highway illumination In this field of illumination, our vital interest is not in the detail of an object, but is there an object in our pathway which will provide a hazard Information on this matter is being gathered as rapidly as possible

HOW MUST SODIUM VAPOR ILLUMINATION BE APPLIED AND WHAT ARE ITS VISUAL ADVANTAGES?

The methods of discernment of an object upon the roadway regardless of the quality of illumination still will fall into four classifications (see Figure 5)

1 Discernment by direct illumination by adequate light falling upon the object
2 By silhouette of the bulk of the object against a lighter background

3. By discernment of revealing glints from polished surfaces on the object.
4. Discernment of the object's position by the shadow it may cast.

From the economic consideration of highway lighting which necessitates relatively low intensities at this particular day, the value of the silhouette effect is paramount. In order to obtain this desirable silhouette effect, we must produce road surface brightnesses of such values and in such positions so that an object between the light source and the driver will be adequately revealed. This desirable condition is brought about by installing the light sources over the traveled road-

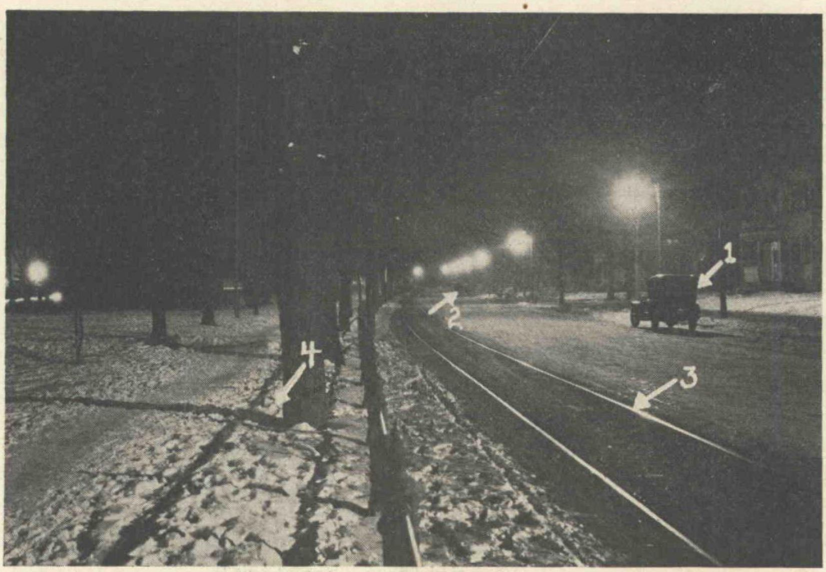

Figure 5. Four Methods of Discernment—Direct Illumination, By Silhouette, Revealing Glints, By Shadow.

way at such a height to minimize glare and close enough together to provide an adequate brightness area above the horizontal to assist in overcoming the partial blinding effect of automobile headlights approaching the driver. In practice excellent results are obtained by use of specified positions as shown in Figure 6. A further factor which has not been given due consideration in the past is the placement of the light unit in such equipment as to provide a portion of the light on the road shoulder. This is essential for two reasons:

1. For illuminating any hazard which may be there or any object which may become a hazard, as a pedestrian stepping across the road (Figure 7).
2. To provide an area of brightness in which the eyes harassed by oncoming light may turn for relief.

This necessity of producing adequate road brightnesses brings up questions concerning contrasts—both color contrast and brightness contrast—as well as those concerning the advantages of various road

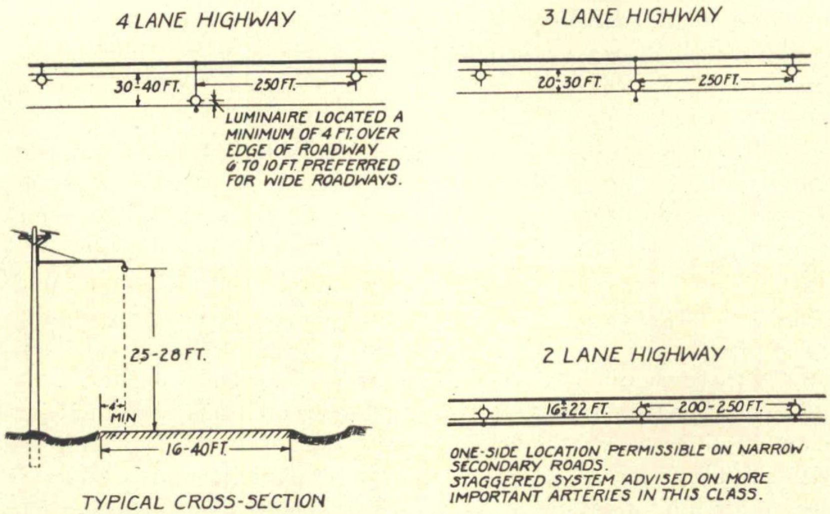

Figure 6. Diagram Showing Most Effective Placement of Highway Lighting Equipment.

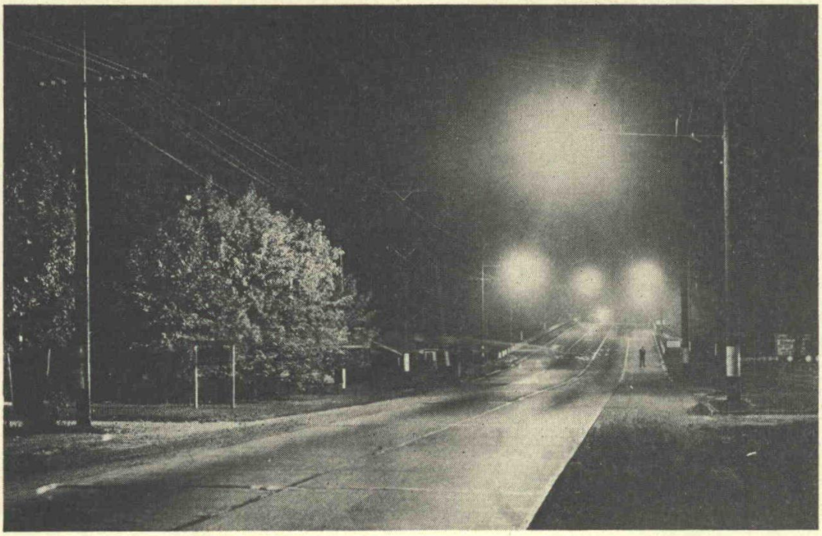

Figure 7. Most Effective Equipment Placed in Most Effective Manner Reveals Pedestrian or Object on Road by Silhouette.

surfaces ranging from new, clean, light colored cement surfaces to black bituminous surfaces.

Color contrasts generally cease to exist under sodium vapor illumination and the advantage of color contrast, which is generally considerable, disappears We must, however, discriminate between the advantages of color contrast and those of color discrimination The latter in reality should not be considered because we are dealing with low levels of illumination at which point the eye in any case is not accustomed to distinguish color due to the fact that the illumination is so low that the cones of the retina are out of action and we are using only rod vision

In studying the fundamental factors regarding vision it has been found that contrast is the most generally effective factor in seeing It has been found that in intensity variations ranging from low values of artificial light to those of full daylight, the minimum size of an object which is just visible varies through a range of 3 1 With brightness contrast variations, however, this range in minimum size becomes 30 1 The brightness contrasts studied were those of very low value, to the highest which is represented by black on white

Attempting to get accurate measurements on visual effects is just about as difficult as attempting to measure the length of a race track with a rubber band Therefore, the careful experimenter is reluctant to give forth information until often-times the number of observations runs into the hundreds of thousands However, due to the intense interest that has been displayed in sodium vapor illumination, Doctor Luckiesh and Mr. Moss have released some preliminary figures which may indicate the necessity for caution in making any statement which in the light of future developments may become rash In order to study the influence upon contrast of sodium illumination one hundred and one specimens of colored paper, representing hues throughout the spectrum, including the purples with their varying tints and shades were selected They were studied under equal intensities of illumination from sodium vapor lamps and tungsten filament lamps After obtaining the reflection factors of these one hundred and one specimens, the determination of contrast of five thousand pairs becomes merely a mathematical procedure. We define contrast as the ratio of difference between the two brightnesses, divided by the higher brightness A perfect black on a perfect white is a contrast of 100 per cent Forty-two of the specimens under sodium lighting appeared brighter than under tungsten lighting and had an average reflection factor under sodium of 53 and an average under the tungsten of 45 Forty-two of the other specimens appeared brighter under the tungsten illumination, having an average reflection factor of 29 compared to their average reflection factor under sodium of 22. Seventeen of the specimens were approximately the same with an average reflection factor of 27

Of the five thousand pairs that would be possible, combining the 101 specimens, report has been made upon the study of 595 This number has been sufficient to show again the general superiority of sodium light over tungsten light in accentuating contrast For example, 267 pairs exhibited greater brightness contrast under sodium light than under tungsten light The average brightness contrast of these was 50 per cent under the tungsten light and 66 per cent under sodium light One hundred and fifty-two pairs exhibited lesser brightness contrast under sodium light than under tungsten light The average brightness contrast of these was 43 per cent under tungsten light and 29 per cent under sodium light One hundred and seventy-six pairs exhibited brightness contrast varying within plus or minus five per cent so they were not included in the foregoing in which the difference in brightness contrasts was greater than five per cent under both illuminants Such low contrast cannot play an important part in seeing They are so low that quick and uncertain seeing must rely upon other factors

In the problem of street and highway lighting, the question of road surfaces is coming more and more to the fore It has only been during the past two years that lighting men in general have come to a realization of how dependent we are upon road surfaces for adequate or inadequate visibility Mr C A B Halvorson's discussion of this problem is reported in the Eleventh Proceedings of the Highway Research Board, Part I, Page 399

Three of the types of road surface in common use, on roads having sufficient traffic to warrant highway lighting, are portland cement concrete, bituminous macadam and sheet asphalt These three are entirely different in their characteristics and act differently under fixed and portable lighting

Visibility under street and highway lighting is, of course, an entirely different matter from that in the home or office Under the ordinary lighting on our public thoroughfares, we are seeing at such low levels of illumination that it is impossible to see by direct light reflected from the object For example, an object in the roadway may have from 0 05 to 0 10 foot-candle, falling on it, but with the object itself having a reflection factor of not more than 50 per cent and presumably as low as one and two per cent The apparent intensity on the object is of the order of 0 001 to 0 05 foot-candle which is practically unnoticeable Accordingly we are forced to rely on silhouette vision, in which foot-candle intensity plays no part The only requisite for silhouette visibility is a lighted background against which the object can be seen in relief From the physical aspects of the case, the lighted background must consist of the road itself and as such the sharpness of the silhouette is determined by the brightness of the road surface, and, as present tests may indicate, the spectral characteristics of the illuminant

Each of the three types of road surfaces mentioned above has different characteristics under light In the first place they reflect light differently Sheet asphalt acts as a mirror or a water surface reflecting light in streaks or specularly Bituminous macadam reflects as a blotter does or diffusely scatters the rays in all directions. Portland cement concrete also reflects diffusely but the dispersion varies according to the roughness of the surface However, both of the last two pavements reflect specularly when they become wet The reflection factor for portland cement concrete is about 80 per cent while it is around 4 per cent for bituminous macadam and asphalt

Since visibility under street and highway lighting is dependent on the brightness of the road surface it is interesting to compare the brightnesses of the three surfaces Sheet asphalt possesses the highest brightness when the roadway viewed lies between the observer and the lamp, but the brightness (4 per cent of the light) is confined to a strip very little wider than the light source itself, although the strip may be more than a 1000 feet long Thus if an object happens to be in line with the strip of brightness it is quite easily seen, while if it is to one side, it is as if in total darkness

Bituminous macadam, reflecting diffusely, gives only a circle of relatively low brightness directly under the lamp since the 4 per cent reflected from it is dispersed in all directions Accordingly a very large lamp is necessary to give a high brightness on such a road surface

Portland cement concrete likewise disperses the light in all directions, which is an advantage in itself since the entire width of roadway thus becomes a background instead of the small strip of brightness present with asphalt or any pavement when wet However, due to the high reflection coefficient of concrete, its brightness is high with a relatively small source of light

Directly connected with this problem also is the action of these surfaces under automobile headlights What is said above about the action of bituminous macadam and portland cement concrete under fixed lighting is also true of them under portable lighting since the driver is dependent on the amount of light reflected back to his eyes for vision The headlights of oncoming cars should be less glaring on portland cement concrete than on a black road since the road surface is brighter making the contrast less marked On asphalt pavement, as on all wet pavements, the light from headlights is reflected specularly away from the driver where it does little good

It appears therefore that of the three types discussed portland cement concrete surfaces are the best and most economical from the standpoint of lighting The objection that is sometimes raised because of sun glare can be surmounted by the use of sun glasses

It is often proposed that with correctly adjusted headlights street lighting would be unnecessary It may be argued however, that since under headlights, vision is dependent upon reflected light, under adverse conditions if the object is dark, having a very low reflection factor, there will be little reflected light and, therefore, no effective seeing Likewise under oncoming headlights, the iris of the eye is contracted and unless the road surface beyond the opposing headlights is bright, vision is too greatly handicapped for safety The question of balance between the illumination of headlights and street lights causing an object of certain color to disappear has also been raised This may happen particularly on very uniformly lighted streets, a method of lighting which is not the best practice With non-uniform lighting, such a merging would be more difficult, and furthermore if the roadway is adequately lighted, there is no necessity for headlights of high enough intensity to cause such a balance

WHAT ECONOMIES MAY THE FUTURE HOLD FOR SODIUM VAPOR ILLUMINATION?

The peak of the relative luminostic curve of the eye is at the wave length of 5550 Angstrom units If all of the energy supplied to a luminous tube were emitted at this wave length the luminous efficiency would be in the neighborhood of 668 lumens per watt In the sodium lamp of today we find that the energy emitted in the infra-red line is about one-quarter of that emitted at the yellow D line The summation of the remaining lines both in the visible and the ultra-violet region only amounts to a few per cent The majority of light emitted from our sodium lamps is of the wave length of 5890 Angstrom units or about 78 per cent as luminous as the light at the peak of 5550 Angstrom units

The heating losses, plus those which are incidental to the use of governing equipment due to the arc characteristics of the sodium lamps point to the approximate theoretical maximum of 320 lumens per watt The practical limit that has been reached so far has been 70 lumens per watt The discrepancy of the efficiencies shown by these two figures is accounted for by the losses in the tube due to heat production and heat radiation and generative equipment It is very apparent from the progress made so far that we still have a long way to go in the producing of practical efficiencies higher than we have to-day The efficiencies that are being obtained in practice in the demonstrations throughout the country are of the order of 40 lumens per watt

Conservative or cautious as I may appear to be in speaking of the superiority of sodium for seeing on the highway, we still can become enthusiastic for the economies that may accrue through knowledge gained by greater experience Certainly we can become most enthusiastic in considering the potential increased luminous and visual efficiencies, and the lessened cost of generative equipment and distribution equipment which undoubtedly will be lowered through the application of engineering ability and mechanical ingenuity.

The welfare of human beings is the ultimate objective of lighting whether the majority of us recognize this or not Intelligent and skillful use of various aids for seeing is necessary The development and use of sodium vapor as an illuminant for our roads is but the outgrowth of the use of our past incandescent sources. Any discussion of economies that may be effected in the future must of necessity give due weight to the advantages and economies that accrue to the individual citizen. It must be conceded that the benefits and economies of scientific lighting are best revealed by controlled laboratory research.

Controlled laboratory research, in studying the behavior of automobile drivers upon the highways, either with or without the benefit of adequate and properly applied highway illumination is difficult. Individual accident reports as filed with the authorities may not be relied on as being 100 per cent accurate, particularly in the reporting

Figure 8. Daytime View of Intersection of New Jersey Route No. 6 and Parsippany Boulevard.

of how the accident occurred. However, a great number of reports covering a considerable period of time, may be relied upon as being indicative, or giving a close approximation of the actual history of the accidents on the highway. Reductions ranging from 30 to 60 per cent of night accidents have been reported in installations where accident history has been available before and after installing highway lighting.

A most interesting study is being made in New Jersey at the present time in conjunction with Commissioner Hoffman's Safety Educational program. Heavily traveled sections of road throughout the state were chosen for study. These sections were each one mile in length and were chosen for their known bad accident record as well as their type of road surface, topography of adjacent country and their inherent accident hazards.

Two such sections of highway have been lighted for six months with incandescent lamps and modern efficient equipment. One section is on New Jersey State Highway No. 6 at and adjacent to Parsippany Boulevard, which intersection is about four miles east of Dover, New Jersey. One portion of the section is bituminous macadam and the other Belgian block. This is a two-lane highway, with many curves and slight grades, due to the rolling character of the country in northern New Jersey. As generally found in New Jersey there is a narrow gravel shoulder averaging six to eight feet on either side of the paved portion of the roadway. Day and night views of the intersection are shown in Figures 8 and 9.

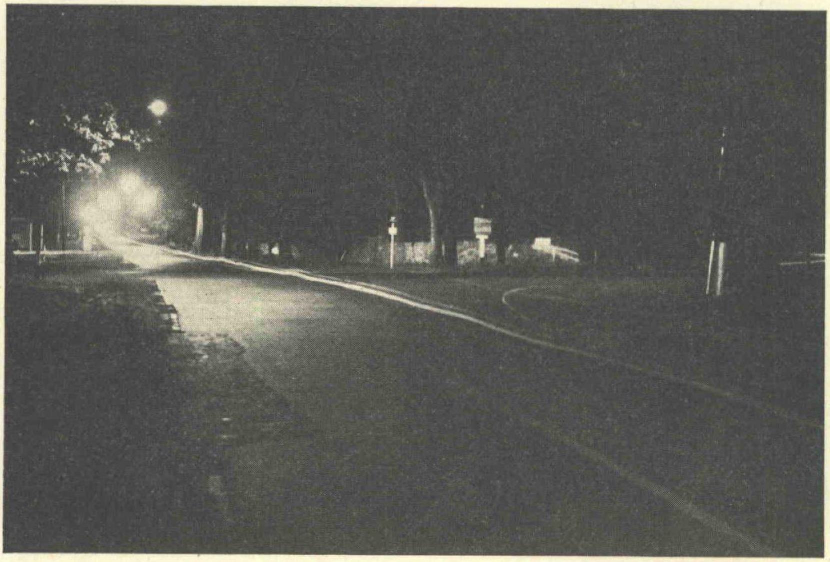

Figure 9. Safety Demonstration Lighting at Intersection of Route No. 6 and Parsippany Boulevard. Note illumination of entire roadway and shoulder.

The second section is in South Jersey where we find the characteristic of flat country. This highway is two lanes in width, of concrete construction, with an average of 15 feet of light gravel shoulder on either side of the paved portion of the roadway. Night and day driving conditions are clearly indicated in Figures 10 and 11. Six thousand lumen incandescent lamps are used in the highway proper and the illumnation is built up at the intersection by employing 10,000 lumen incandescent lamps. Proper seeing conditions are provided by the employment of efficient refracting globes. Comfortable seeing conditions are produced by properly designed prisms in conjunction with a rippled effect on the outer surface of the clear glass globe, and the use of proper mounting height, spacing and road overhang, as discussed previously.

The accident history of New Jersey as reported in 1932 includes 45,867 accidents causing 31,426 injuries and 1,180 deaths. It is true that there is a concentration of traffic hazards in New Jersey and that there are more miles of improved and heavily traveled highways per

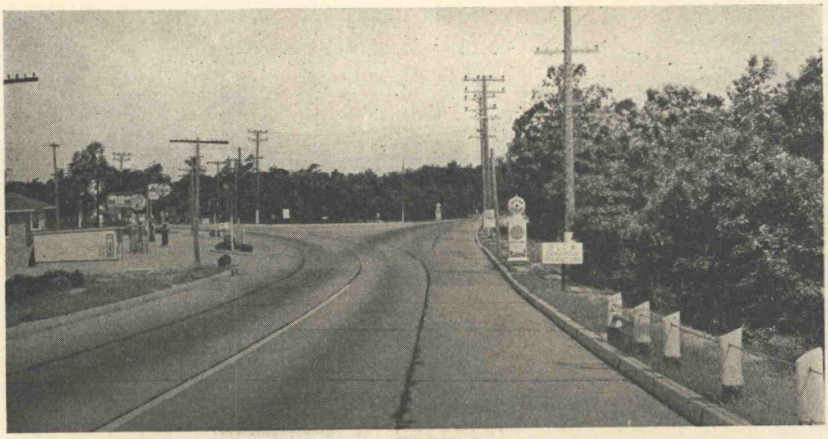

Figure 10. Safety Demonstration Lighting at Intersection of Routes Nos. 33 and 34 in New Jersey. Daylight.

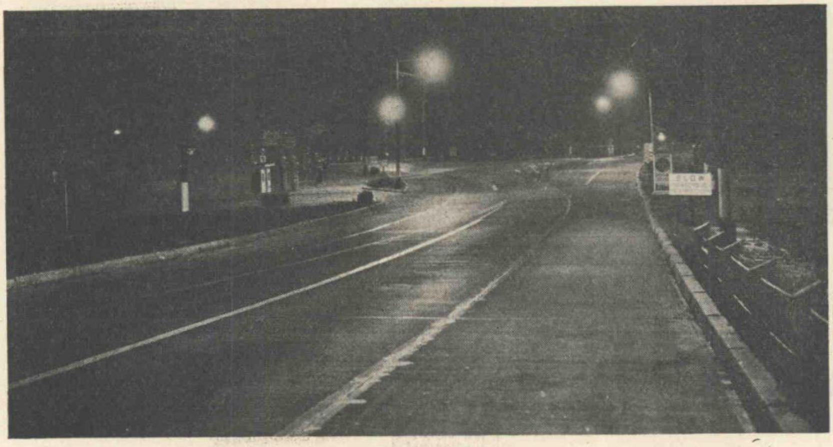

Figure 11. Safety Demonstration Lighting at Intersection of Routes Nos. 33 and 34 in New Jersey. Night. Daytime sight distances are provided through engineered illumination.

unit of area than may be the average throughout the United States. Nevertheless, every state has many miles of similarly heavily traveled highways which should be studied in order that the elimination of hazards and accidents may be brought about quickly and surely.

All of us who drive at night realize the immediate feeling of relaxation and safety upon entering and driving upon a well-lighted stretch of road. It is difficult indeed to put a dollars and cents value upon that sense of security and freedom of the mind from the worries incident to driving without adequate vision. It is not so difficult, however, to quantitatively appraise the losses which are encountered by the general public through the lack of good vision at night. It is estimated by those who have made a study of the accident statistics and the accident causes throughout the country that nearly one-half of the total accidents that occur upon our streets and highways at night are caused in the main by inadequate vision through lack of illumination. At the same

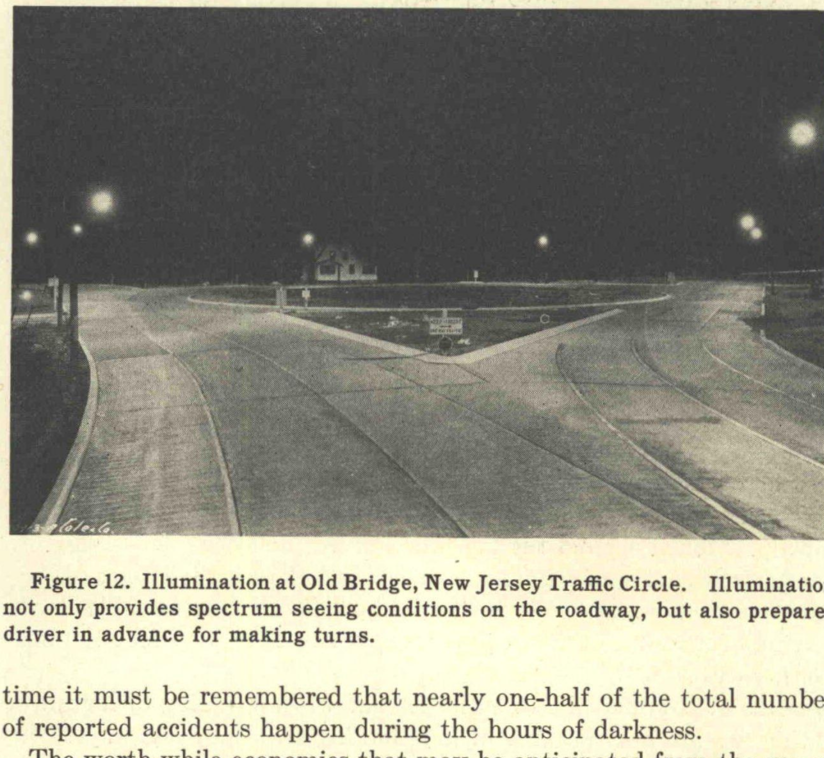

Figure 12. Illumination at Old Bridge, New Jersey Traffic Circle. Illumination not only provides spectrum seeing conditions on the roadway, but also prepares driver in advance for making turns.

time it must be remembered that nearly one-half of the total number of reported accidents happen during the hours of darkness.

The worth-while economies that may be anticipated from the use of this or any modern illuminant cannot adequately be measured by the luminous efficiency of the source but must be measured by the degree in which the illuminants are put into actual use upon our roads and highways. The saving of human lives, the prevention of suffering due to pain of physical and mental hurt resulting from loss of our loved ones, the relaxation of nervous and muscular tension so essential in these days of high speed living, are all economies. The degree to which they may add to our sum total of economies largely depends upon the highway engineers, who study this problem and make proper recommendations in the true light of their findings.

DISCUSSION

ON

HIGHWAY LIGHTING

Mr C C Ahles, *New York State Highway Department* I observed the lighting at Schenectady and I thought you might be interested in the cost It is quite an item, this cost of lighting highways it is, in general, a difficult thing to put down as a lump sum but if highways were lighted so that it could be done upon a real production basis, the cost would not be more than perhaps $1200 to $1500 per mile per year—that is for real highway lighting

Mr Lowe Mr Ahles' quotation of $1200 to $1500 per mile per year as the cost of highway lighting may be considered fair at the present time These costs, however, have been based upon present street lighting schedules and rates It is difficult indeed to speak of street lighting costs in a general way, as there are so many factors in various localities which might materially change the amount, such as the facilities already present, the proportionate distribution of costs to other types of consumers, the cost of maintenance which varies greatly when considering the number of lights to be taken care of, the width of the right-of-way, necessitating different types of equipment to get the necessary overhangs and mounting heights, the distance between units, energy charges, etc

Motor Vehicle Commissioner Harold G Hoffman of New Jersey has said "It is true that proper and sufficient highway lighting would require an expenditure of considerable money However, this expense, properly weighed against the benefits derived, including the saving of lives and the saving of millions of dollars, all of which is a part of the present tremendous economic loss, would, we feel, unquestionably show a saving of money to the tax payer who must ultimately foot the bill in either case "

One point I would like to stress is that when speaking of highway lighting, we are not speaking of illuminating all of our highways It is only where there is economic justification for lighting It is difficult indeed for the average man to realize the point of economic justification for the expenditure of the money necessary for adequate and proper highway lighting This latter problem may be solved by analysis of accident records collected by motor vehicle departments, safety organizations, police departments, etc

REPORT OF JOINT COMMITTEE ON ROADSIDE DEVELOPMENT

HIGHWAY RESEARCH BOARD AND AMERICAN ASSOCIATION OF STATE HIGHWAY OFFICIALS

LUTHER M KEITH, *Chairman*

Director of Roadside Development, Connecticut State Highway Department

SYNOPSIS

The objectives of roadside development are the conservation, enhancement, and display of the natural beauty of the landscape and the provision of maximum safety, utility, economy, and recreation facilities

The Committee believes that roadside development should be considered an essential part of highway design, construction and maintenance and recommends That state highway department personnel should include a person competent to carry on this work, that highway authorities should control the right-of-way and its appurtenances and have power to acquire adequate rights-of-way and other parcels of land for the public benefit, that a definite part of highway funds should be budgeted for roadside development, that highway authorities should cooperate with local organizations interested in roadside work and that state nurseries should in general not grow stock obtainable from commercial nurseries

There is an appendix giving digests of roadside development laws for several states and a list of references

INTRODUCTION

In 1932, a joint project of the American Association of State Highway Officials and the Highway Research Board was organized to make a survey of the laws, funds, organizations and technical practices relating to roadside development

This progress report does not attempt to cover all the details of roadside development It defines the objectives of roadside development and discusses means of reaching them The Committee expects to collect and present further information on some phases of the problem in a future report

Appended to this report is a digest of information received from several States on their laws, funds and methods of administration for roadside development, and a list of references pertinent to this subject

GENERAL DISCUSSION OF THE PROBLEM

From a study of the available information, it appears that the fundamental principles of roadside development are well known to the people who have worked extensively in this field However, the general public and many highway officials who have not heretofore been concerned with this matter have given the problem little thought That does not mean that the citizen or tourist will not appreciate anything which is done to make the highways more pleasant and useful The ordinary motorist may not be vocal in his desire for better looking highways, but he does not need a course in Landscape Architecture to be able to appreciate consciously or sub-consciously, an unmarred landscape or a pleasant shady place for a rest along the road Nor will he have any regrets for the elimination of the many eye-sores such as billboards and signs, uninviting camps, hot-dog shacks, ugly filling stations, automobile graveyards and trash dumps

NEED FOR EDUCATION

Many of the most important items of roadside development cannot be initiated until legislative action is taken For this reason, the people must be informed as far as possible of what is needed In order to make satisfactory headway in some phases of the work, definite allotment of funds must be made for the purpose Before any such funds are furnished, it is necessary to have the support of the taxpayers Hence it is necessary that the public be familiarized with the aesthetic requirements and possibilities of highways The educational work that is being carried on by highway authorities, Women's Clubs, Garden Clubs, Legion Posts, the American Nature Association, the American Civic Association, and the National Council for Protection of Roadside Beauty and others shows that many people are willing to cooperate even to the extent of financing some betterment work when they understand what is needed

In California, some 700 miles have been planted with trees and maintained for the first year by public spirited individuals and organizations The same sort of work has been done in several other states and has been of great value in furnishing the general public with concrete examples of what can be done to improve the roadside appearance Many opportunities to do this lie in the improvement of business structures along the way It has been found that many owners of such establishments are willing to improve the appearance of their property but they are in need of ideas of how to accomplish it A trade journal is now being supplied to wayside business men by the Art Center of New York for the purpose of helping them to better their businesses through improvement of their property

It is essential that highway engineers themselves become more familiar with roadside development. Some states have made notable progress in this branch of highway work Others have done very little, perhaps because of lack of public support or because the need to change prevailing practices has not been apparent Especial impetus has been given this year to roadside development by the ruling of the Public Works Board under the Industrial Recovery Act Six items of highway work are to have priority and the second one named is "appropriate landscaping of parkways or roadsides on a reasonably extensive mileage" A further ruling (October, 1933) of the Public Works Administration stipulates, by resolution, that insofar as practicable and feasible the rights-of-way for highways built with loans or grants from public works funds after January 1st, shall be at least 150 feet wide to accomodate foot paths and screening by planting trees and shrubs

In order to properly execute roadside development work, the personnel of highway organizations must be educated in the economics, practices and increasing importance of this work The first essential is that each organization should have a competent person with authority to design and carry out roadside development work

DEFINITION OF OBJECTIVES

Roadside development must conserve, enhance and effectively display the natural beauty of the landscape through which the highway passes as well as provide maximum safety, utility, economy, and recreation facilities by means of proper location, design, construction and maintenance of the highway

CONSERVATION OF NATURAL BEAUTY AND ADVANTAGES

To conserve the landscape is to adjust and adapt the highway so as to make maximum practical use of the landscape without unnecessarily disturbing the balance of nature This is accomplished by preservation of desirable native growth, outstanding topographical features such as water courses and geological formations, scenic values and historic sites

Designs should be varied according to the kind of road (parkway, commercial or recreational) and will be affected by the location, width of right-of-way, alignment and cross section Standards will also vary with the regions they serve, some being suitable for open country and others for suburban areas Near the larger cities, there are opportunities for parkways for passenger traffic only. Less expensive design must be used near most cities and towns but it should be remembered that the better the highway, the farther out the urban area will spread Most of the highway mileage will be in open country and elaborate designs are not necessary or justified The requirements of these roads will be met, in large measure, if adequate rights-of-way are provided and the abutting lands are protected from unsightly encroachments

Location enters into the design only on those roads on new routes or radical relocations If the road is to carry a large amount of commerical traffic, the location should minimize the depreciation of residence property In many sections of rough country, a new location

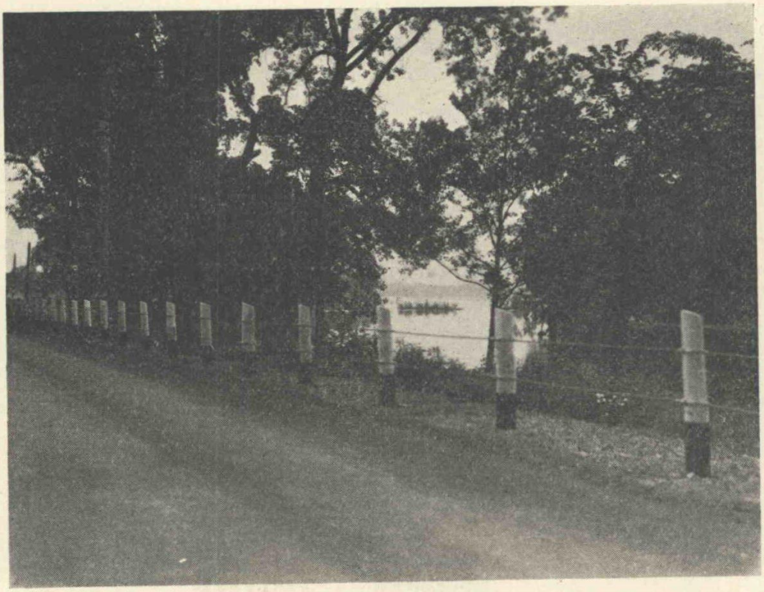

Figure 1. This picture illustrates the method used by the Connecticut Highway Department in opening up vistas adjacent to rivers in which it is the policy to offer to the motorist frequent glimpses of the water rather than one continual panorama.

Figure 2. Scenic View opened along Susquehanna River in Pennsylvania

away from the old crooked roads will not involve destruction of trees and old structures toned to the landscape Location to fit the landscape is tied up with width of right-of-way, alignment and cross section The right-of-way must be of sufficient width for the proper cross section and also for future planting In wooded territory, extra width will preserve strips of timber of great value and utility Alignments both in plan and profile offer design opportunities not usually recognized except by men trained to see their effects on appearances Curves rather than tangents fit a rolling topography Long tangents may possibly be justified in flat country although excessive length invites operating hazards due to boredom The design should minimize the scars of construction While it is desirable to have a limited number of standard cross sections, such standards should be flexible enough to fit special problems Flat slopes rounded into the natural surface will look better and be easier maintained Many times a good tree can be saved by a slight change in cross section or alignment

Too often the highways are built where the aim seems to be to lay down a series of geometrically exact figures of cut and fill on the face of nature A modification of this plan might fit unobtrusively into the landscape at no greater expense even though some increased cost might well be justified If it is worth while to spend money on embellishments of highways such as plantings of trees and shrubs it is important to plan the alignments and cross sections with due regard for their effect on appearance

Construction practices should be revised if necessary to preserve trees or other features worth saving In many cases, the fertile top soil from cuts is buried in the bottoms of fills leaving only sterile soil on the slopes where the maintenance crew tries vainly to grow sod to prevent erosion At no great expense, the top soil could be saved for the slopes In timbered country, some trees would be worthy to stand inside the cleared area without harm to the safety or use of the road Is it necessary to hew exactly to the inch and clear everything between two arbitrary lines? The resident engineers and contractors can find many ways to improve the new road's appearance if they are given the instructions and authority to do so All construction débris and material should be cleaned from the right-of-way and its vicinity

After construction is finished, regular maintenance will be necessary to preserve the good appearance of the roadside The natural growth trees and shrubs will need trimming and some tree surgery will be needed for injured or diseased trees Spraying may be needed to control insect pests New plantings must be cultivated and perhaps watered In some states such cultivation may be necessary for as long as five years Weeds or other noxious growths must be cut Trimming of trees may be required for telephone or power lines Rubbish must be removed Recreational facilities such as roadside parks, camps and drinking fountains must be maintained in a neat manner.

ENHANCEMENT AND DISPLAY OF THE LANDSCAPE

The enhancement of the landscape is achieved by properly designed plantings to enrich existing growth, to relieve monotony and add interest, to cover and screen unsightliness and to create and frame attractive vistas.

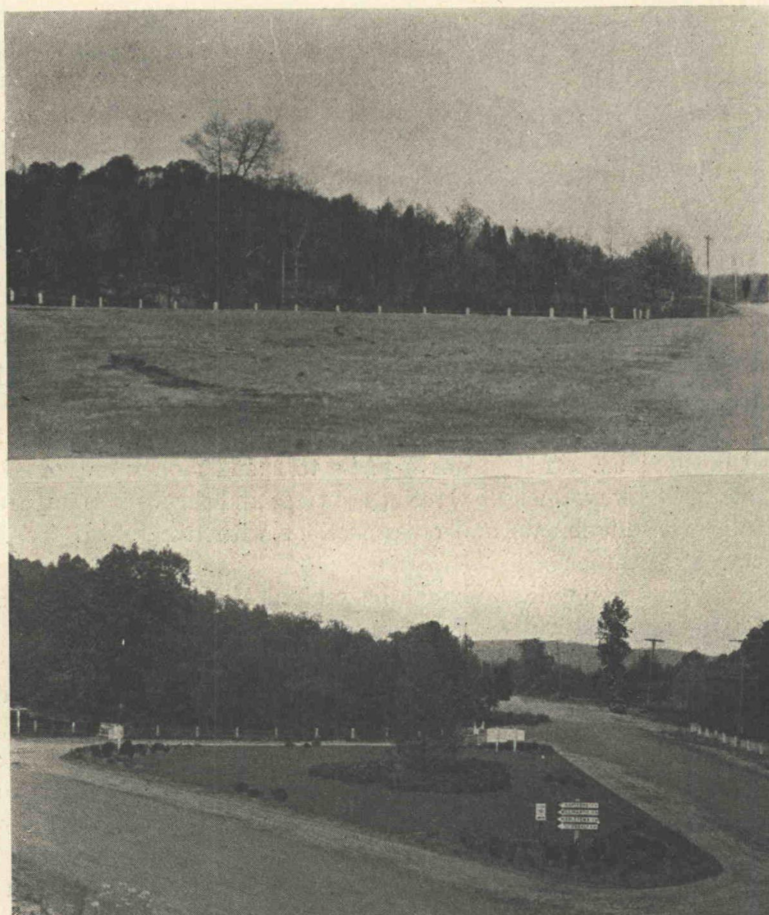

Figure 3. The upper view shows a triangular intersection as left at the completion of a new road in the Town of East Haddam, Conn. The lower picture shows how this eye-sore was developed into a highway garden by covering the subgrade with a good quality of loam, grass seed, dwarf evergeeens, mountain laurel and a European Larch Tree.

All planting should be harmonious and adapted to the locality and should usually be native stock indigenous to the locality. The use of native stock has three distinct advantages. First, native plants harmonize with the plant character of the surrounding country and preserve its individuality. Second, native plants are hardier under roadside conditions than are imported or developed plants. Third, native plants require less maintenance, an important factor in any state-wide program. In some cases, it is desirable to use non-native stock as an addition to the predominating native species. Trees may be set in groups or in rows although, as a rule, the group planting is preferable. It is important that the width of the right-of-way should be such that any planting will be reasonably free from future disturbance. This will require foresight as to the possible later development along the highway.

Locations usually most in need of plantings to improve appearance are grade separations, approaches to cities and towns, slopes of cuts

Figure 4. Planting of Oriental Planes, Pennsylvania

and fills, bridge approaches, intersections, and state entrances. Pole lines and other unsightliness along the highway may be screened.

SAFETY, UTILITY AND ECONOMY

Roadside development in several of its phases adds to the safety, utility and economy of the highway. Flat slopes from the shoulder to the ditch and on fills of less than five or six feet permit seeding and are less dangerous to traffic than steep slopes. Flat slopes with uniform sod are easier to mow and maintain and less subject to erosion. Where it is possible to use flat slopes on fills, less guard rail will be needed. In rough country where rock is available, rock guard rail may be used for better appearance and greater safety. Back slopes of cuts should be planted to improve appearance and control slides and erosion. Erosion by streams can, in many cases, be controlled by tree planting. Formal plantings may be used to designate certain locations such as schools, intersections and railroads. Trees can be planted on the out-

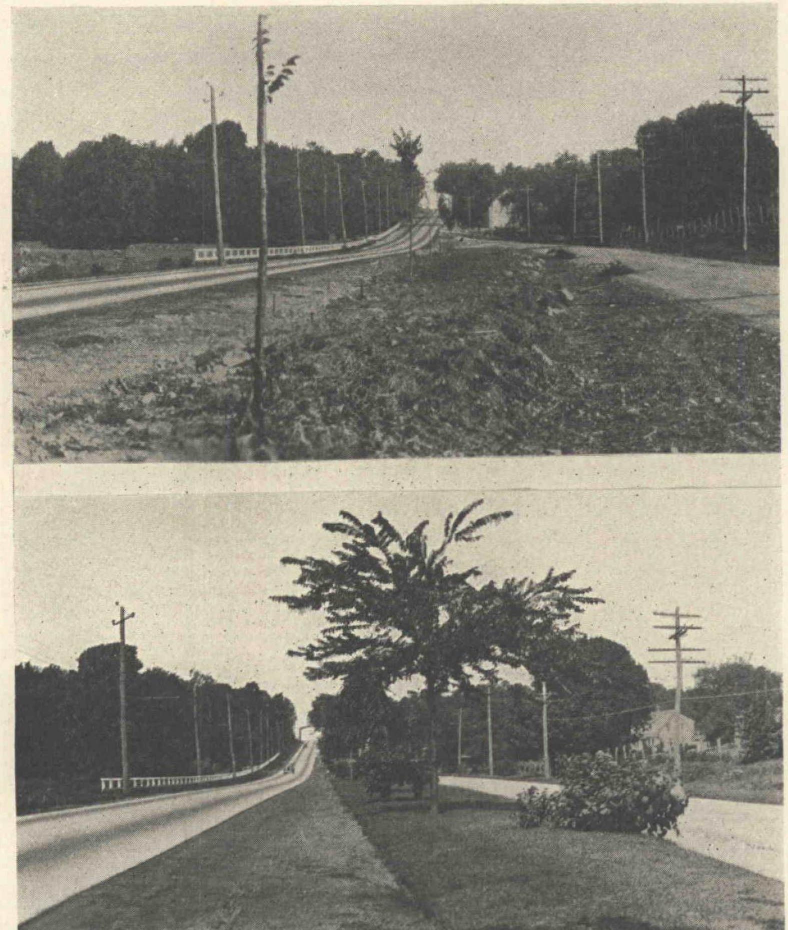

Figure 5. The upper view shows the unsightly island area created at the time the new concrete road was constructed in the Town of Montville, Conn., and the old roadbed abandoned. The after effect may be appreciated in the lower picture of the same location.

side of curves and sharp turns or corners to indicate change of direction. Where sufficient right-of-way is available, prevention of snow drifts in the highway may be accomplished by planting hedges or trees. Parking places, even though small, where the traveler may stop are of great utility; where feasible, these should be landscaped and equipped with drinking fountains. Where laws permit, the location and construction of pole lines should be controlled to give minimum interference with the use and appearance of the highway.

RECREATION FACILITIES

Along many highways, there are no places where the traveling public may stop except along the shoulder of the traveled way or upon private property. Since a large amount of travel is for purposes of recreation, there should be convenient stopping places on public property such as picnic grounds, highway parks, drinking fountains, turn-outs and lookout points. Such areas need not be large nor expensive. On some roads, stopping places may be available in naturally wooded areas; on others, some planting will be desirable. In many places, there are small tracts between old and new locations which should be retained or acquired for the benefit of the public and to prevent their use by private business. Some small areas may even be acquired by gift from the owners. Many rights-of-way are wide enough to permit turn-outs or small picnic grounds to be built.

Figure 6. White willow posts and logs to protect slope three months after planting, Pennsylvania.

COMMITTEE RECOMMENDATIONS

1. Every road-building agency should include a person competent to design and carry out roadside development work. His work should be

considered an essential part of the design, construction and maintenance

2 Absolute control of the highway right-of-way and all its appurtenances should be vested in the highway authority

3 Highway authorities should have power to acquire adequate right-of-way for present or future roadside development They should also be empowered to keep or acquire title or easements in strips or parcels of land along the highway for the benefit and enjoyment of the public

4 Highway authorities should budget a definite part of their funds for roadside development and its maintenance

5 There should be cooperation by the highway authorities with individuals, organizations and local communities interested in roadside development

6 This Committee endorses the following resolution of the Roadside Development Committee of the American Association of State Highway Officials "The Committee further recommends the establishment of State Highway Department nurseries only for the development of salvaged or collected native plant material, for the storage of surplus purchased plant material, and for the growing of such stock as is not obtainable from commercial nurseries "

PROJECT COMMITTEE ON "ROADSIDE DEVELOPMENT"

In Cooperation with the American Association of State Highway Officials

Chairman—Luther M Keith, Director of Roadside Development, State Highway Department, Hartford, Conn

Research Assistant, W V McCown

Appointed by the American Association of State Highway Officials

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Ludwig, Walter D , Highway Forester, Pennsylvania Department of Highways, Harrisburg, Pennsylvania

Moorefield, C H , State Highway Engineer, State Highway Department, Columbia, South Carolina

Neale, H J , Landscape Engineer, Department of Highways, Room No 1 State Capitol Building, Richmond, Virginia

Appointed by the Highway Research Board, National Research Council

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Aust, Franz, Associate Professor of Landscape Design, Department of Horticulture, The University of Wisconsin, Madison, Wisconsin

Cutler, T. H , Chief Engineer, Missouri State Highway Department, Jefferson City, Missouri

Appendix I

Laws and Funds for Roadside Development

Following is a digest of information received from states active in roadside development work concerning their laws, funds and administrative practices for roadside development

California In general, the work is done by the Division of Highways under its authority to construct and maintain state highways In 1931, the Highway Commission was given authority to secure lands adjoining highways for public parks, and also lands and trees within 300 feet of the center line on each side of any state road, for culture or support of trees when such acquisition is an aid in maintaining or preserving the roadbed or an aid in the maintenance of scenic beauties. Under an act passed in 1933, provision is made for carrying on as maintenance such general utility services as roadside plantings The funds are from specific maintenance moneys, not detailed in the budget Work is financed as projects develop and funds become available Much help has been given by individuals and organizations who deposit sufficient funds with the department to plant and maintain trees for one year The work is handled by the regular maintenance organization, T. H. Dennis, Maintenance Engineer, and an Arboriculturist who supervises the work throughout the state He advises district maintenance engineers and superintendents and prepares plans for particular projects

Connecticut The law requires any person to secure a permit from the highway commissioner to remove or prune any tree, shrub or vegetation in the right-of-way. The Commissioner may plant in the highway or on adjoining land by agreement or by condemning easement Funds are budgeted from state highway funds In 1932 $425,223 were spent for planting, maintenance, mowing, construction of gardens, picnic grounds, etc The Bureau of Roadside Development, L M Keith, Director, has supervision and maintenance of everything except drainage, on the roadsides outside of the outer gutter edge and of waste areas The work includes mowing, removal of trees, maintenance of slopes and embankments, planting, seeding, maintenance of picnic grounds, etc

Illinois. The Department of Public Works may issue permits for planting of trees, shrubs and flowers to persons, associations or societies The Department prepares the ground, supervises the planting and maintains it with regular forces The work is largely of an advisory nature All trees, shrubs, etc, are protected by law Funds to care for and maintain plantings are taken from the Maintenance budget of the highway department

Maryland Since June, 1931, Maryland has been deriving revenue for roadside development work from the licenses and fees required for outdoor advertising along the highways The maintenance department has removed some 23,000 unlawful signs, planted 3,480 trees along both sides of 30 miles of highway, planted 18,000 vines and plants on slopes, sodded 22,300 sq yds of slopes, and seeded about four acres of park area Frank P Scrivener, Jr, Maintenance Engineer, is in charge

Massachusetts In 1921, an office was created to be filled by one with special training in landscape planting to "beautify the state highway roadsides" The program has since progressed with definite aims and accomplishments Planting is done under the maintenance engineer and is supported by a separate allotment of funds for that specific purpose

Michigan Trees and shrubs on all highways are protected by law It is the State Highway Commissioner's duty to plant trees along state trunk and state reward roads, with the consent of the owners of adjoining property Money is budgeted from construction funds for development on new trunk lines Money for yearly roadside maintenance is budgeted from general maintenance funds The work is in charge of a Landscape Forester

Minnesota The Commissioner of Highways designates the necessary width of right-of-way One hundred feet is the standard width All highways, roads and trails within forest areas are established as fire breaks The Division of Forestry has authority to remove or clean up any inflammable material for 200 feet on either side of the center line of the fire-break roads Any money used is taken from the highway funds but local groups are encouraged to plant and maintain the plantings An assistant engineer and forester have been assigned to development work, under the joint supervision of the construction and maintenance engineers

Missouri The law provides for construction and maintenance and all work incidental thereto This is interpreted to provide for roadside planting and development Trees, shrubs, etc, on the highway are protected by law No signs or places of business are allowed on the right-of-way Funds are approved for expenditure by the Commission from general highway funds The commission urges cooperation of local groups interested in planting The work is handled by the Bureau of Maintenance An experienced landscape designer is employed

New Hampshire There is no special law to provide authority or funds for roadside work About $6,000 is budgeted annually for Highway Marking and Roadside Development Dependence is placed on cooperative effort with local groups F A Gardner, assistant engineer is in charge of the work

New Jersey The Highway Commission is empowered to plant and care for trees and shrubbery along state highways, or otherwise beautify the highway The money is expended under the supervision of a landscape architect employed by the commission Funds for roadside projects are to be not more than one per cent of the amount expended in the preceding year for construction of highways The appropriation must be approved by the Governor There have been no funds approved the past two years Appropriations for mantenence of projects is made from the motor vehicle registration money

New projects are recommended to the State Highway Engineer for his approval and that of the Commission O A Deakin, Landscape Engineer

New York The superintendent of Public Works may plant, remove trees or trim trees, and may seed or sod within the highway Trees and shrubs are protected by law Special permit is required for signs in Adirondack Park Any highway money may be used for tree or shrub planting, seeding or sodding Lack of funds has handicapped the work

Oregon The State Highway Commission is empowered to acquire by purchase, gift or condemnation, land necessary for the culture of trees and preservation of scenic places adjacent to state highways and for parks and recreation grounds, also to improve, maintain, and supervise the same Trees, shrubs, and flowers on the highway and on private land within 500 feet of the highway are protected The costs are paid from state highway funds The law is administered by the State Highway Commission and the State Parks Engineer

Pennsylvania The law authorizes planting of trees, shrubs, vines and grasses on or along state highways, also the establishment and maintenance of live snowbreaks Highway authorities may, when necessary to construct or widen a highway, remove trees up to four inches diameter, at 2½ feet above the ground, for larger trees they must have the consent of the owner All trees, plants etc , on public or private property are protected by law Money for planting, etc , comes from the regular road fund A large amount of the work is done in cooperation with interested organizations or individuals The work is carried on by a Highway Forester and six Division Foresters, under the direction of the Secretary of Highways

Rhode Island The State Board of Public Roads is empowered to plant trees, shrubs, and otherwise beautify the area within a state highway Trees, shrubs, etc , are protected by law The work is paid for from the general highway fund It is classed as betterment work under the construction item of the budget The work is done under the direction of the Maintenance Engineer

Virginia The law provides for a landscape architect who is a regular member of the Highway Commission staff, to devise methods to beautify and improve the rights-of-way The highway department may make rules for the protection of trees, plants, etc , on the right-of-way On new work, an allocation of funds is made for seeding and planting the right-of-way Maintenance work is paid for from maintenance funds

The State Landscape Engineer, H J Neale, under the Assistant Engineer in charge of Maintenance, makes an intensive study of conditions and makes recommendations to the construction and maintenance departments

Wisconsin Highway authorities may acquire land for highway purposes and it may be used for any purpose deemed for the public benefit. Irregularly shaped parcels and corners along the highway may be acquired Suitable plantings to improve the highways are authorized Trees, shrubs and vegetation are protected by law On new construction or relocations, any roadside work is charged to the project Maintenance of planting is handled by the regular maintenance forces Beautification is made a part of the construction project and plans are prepared by a part-time landscape horticulturist from the University of Wisconsin, under the direction of the Highway Commission, J C Schmidtmann, Vice-Chairman, M W Torkelson, Director of Regional Planning Work is also carried on in cooperation with cities, clubs, etc

Appendix II

List of References

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3 Compilation of Laws of the Several States Relating to Outdoor Advertising U S Bureau of Public Roads, August 30, 1933

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5 Fitting Highways to the Landscape A report by the Section on Architecture of the Commonwealth Club of California In the Official Journal of the Club, November 15, 1932 (A study of factors which affect aesthetic aspects of rural highways)

6 Four Years Along Connecticut Highways, State Highway Department, Hartford, Connecticut Luther M. Keith, Director of Roadside Development (Development of roadside beautification, highway landscaping in Connecticut, conservation and nurseries, shade tree planting, pole line and shade tree inspection) New edition—Seven Years Along Connecticut Highways, April, 1934

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8. Landscape Planning and Planting of Highways, in President's Conference on Home Building and Home Ownership, 1932. Volume I, page 181 (Types of highways Aesthetic design and planting. Restriction of roadside additions Recommends legislation)

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9 Minnesota Conference on Roadside Development and Use St Paul, Minn , 1932 Mimeographed report of the meeting by S Rex Green, Secretary of the Conference Department of Highways (Outline of problems Details of roadside planting)

10 Modern Roadside Development, John C Schmidtmann, Vice-Chairman, Wisconsin Highway Commission Mimeographed paper from the author (Policies adopted by Wisconsin Highway Commission) Highway Research Abstracts No 12—July, 1934

11 Natural Snow Fences, R A Drought, University of Wisconsin "Public Works," August, 1929 (Comparing costs of planted snow fence with temporary fence)

12 Parks and Highway Beautification, Grover C Dillman "American Highways," January, 1930 (Fundamentals, points to be covered by law)

13 Parkway Features of Interest to the Highway Engineer, E W James "Public Roads," April, 1929 (Main features of Bronx and Westchester Parkways)

14 The Practical as well as the Aesthetic Side of Roadside Beautification, Luther M Keith, Connecticut State Highway Department "American Highways," October, 1931 (Controlling factors, benefits derived)

14a Pennsylvania Highways Beautiful, Department of Highways (Suggestions for highway beautification Cooperation with clubs, associations, etc)

14b Report of Committee on Roads and Highways, American Society of Landscape Architects, P H Elwood, Chairman (Enumeration of more important activities of 1933 under State and Federal jurisdictions)

15 Resolution Favoring Roadside Beautification by American Association of State Highway Officials, November, 1930 (Adequate right-of-way Conservation of natural growth Control of right-of-way Competent person in the Department of Highways)

16. Roadside Beautification, Jay Downer, Chief Engineer, Westchester County Park Commission, "Proceedings of Association of Highway Officials of the North Atlantic States, 1931" (Broad principles to be observed)

17 Roadside Development, J M Bennett, Superintendent of Parks and Forestry, Wayne County, Michigan, Published 1929 265 pages (Field work, design, planting materials and procedure, pole lines, comfort stations, parkways)

18 Roadside Plan and Progress in Massachusetts, James H Taylor, Highway Landscape Supervisor "Public Roads," August, 1929 (State Nursery, planting procedure, maintenance)

19 Roadside Planning, P H Elwood, Jr , Iowa State College, January, 1933 Mimeographed paper (Needed development in Iowa and other states) Highway Research Abstracts No 11—June, 1934

19a Roadside Planting and Development Committee Report, C B Andrews, Chairman The Friends of Our Native Landscape President, Jens Jensen, Ravinia, Illinois October, 1932 (Benefits of highway planting Use of native plants stressed Physical aspects of highway development List of trees and shrubs suitable for Illinois highways)

20 Roadside Trees, Frank P Rogers, "Michigan Roads and Airports," February 16, 1933 (Provision for planting since beginning of the state)

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DISCUSSION
ON
ROADSIDE DEVELOPMENT

MR WILBUR H SIMONSON, *U S Bureau of Public Roads* The careful and detailed analysis of the more general problems associated with roadside development, as learned from the experience of those states which have already made considerable progress in roadside work as well as from the systematic observations and research of the members of the Joint Committee, should be very helpful in effectively coordinating work of this character in connection with the present large program of highway construction

The recommendations at the end of the committee's report are particularly important just now when such a large amount of work of this character is being planned and rushed for execution in the 1933–34 planting season The relative order of the recommendations is logical and to be highly commended The following detailed comments aim only to emphasize by repetition a few of the principle points enumerated in the report under discussion

It is extremely important that the services of competent designers be utilized as far as possible in consultation and collaboration to insure the proper and efficient planning of the work, so as to make doubly sure that good and satisfactory results will be obtained The effectiveness and permanency of roadside improvement work as an essential part of highway construction programs depends largely, of course, upon how successfully the details of roadside development are carried out It is vitally important, therefore, that roadside improvement work be thoroughly done during this present program so that the plantings will be successful and come through with relatively small losses of stock Nothing would handicap or more harmfully slow up the present efforts directed toward the proper planting of the roadsides than "dead" trees bordering the highways a season or two hence, like veritable tombstones in a graveyard Careful preparation in the planning and execution of the work in accordance with approved-landscape design and horticultural standards is the best safeguard against unnecessary losses of this kind The best and most lasting results may be assured in the present program of "putting men to work" through the collaboration of landscape architects and engineers and other technical experts It is not only important that plantings be made to live successfully, it is equally important that the attention of qualified designers be used to develop attractively the incidental structures that the motorist finds along the roadside Care should be given to the detailed design of drinking fountains, the appearance of parking spaces, railings, headwalls, sidehill spring outlets and other similar features that the motorist sees and enjoys A good looking and attractive construction does not necessarily cost any more than a poor looking and ugly one

Conservation of the natural assets is emphasized in the report In this connection, true conservation means the careful trimming of trees and the judicious selection of plant growth for removals in an artistic and scientific manner under competent landscape supervision, and not the careless "butchering" of trees and the haphazard cutting of undergrowth along the highways It is well to emphasize here that considerable thought should be paid to this phase of roadside improvement work where clearings are contemplated

Conservation means the saving of topsoil wherever possible during the initial stages of construction for later use in the final seeding and planting operations Conservation means the avoidance of "rigidity," or hard and fast uniform standards in design and during construction that cause unnecessary "scars" and the irreparable losses of valuable tree growth Conservation also means that careful adjustments should be made in the widths of rights-of-way to include scenic spots, groves, or other landscape features where reasonably available

Conservation policies are economic, because they reduce to the minimum the need for later introductions of materials or extra constructions Conservation means the practical use of indigenous types of local materials, such as boulders or native stone for guard rails, the salvage of suitable plant growth in the path of construction operations in advance of such activities when the material proves to be subject to easy transplanning and storage in a temporory nursery.

Conservation may be best secured by the cooperation of all interested organizations Initial harmony in the planning and execution of the work contributes very largely to the final harmony and attractiveness of the results The first recommendation cannot be too strongly stressed In the midst of the rush of work during this present emergency, competent supervision is vital to insure the economic conservation and successful preservation of the irreplaceable natural assets along highways

The other recommendations are essential if adequate control of the highway borders is to be assured for the safety, comfort, and convenience of the public Legislative authority is vital in some states to put the responsibility in the proper place Unified control is necessary to protect highway investments for the benefit of the users of the road collectively, and not for individual interests It is only through such definite responsibility that the state highway departments may ever hope to design, construct, and then control highways for the fullest and most efficient traffic service at the lowest cost The importance of these recommendations is proportionately increased as the widths of rights-of-way are necessarily increased, and as the roadside borders tend to become more fully developed for the use and enjoyment of the public

Roadside improvement is a fundamental part of modern highway design As such, the setting apart of a reasonable portion of available construction funds for appropriate work of this character is a wise policy It is conservatively estimated that approximately 30 miles of roadside improvement plantings may be accomplished for the expenditure of $25,000, the assumed average cost, let us say, for one mile of high-type highway improvement While the expenditures for roadside mileages will vary, of course, under different conditions and localities, it is believed that this statement is reasonably correct as a basis of comparison of the relative value of the roadside dollar when considering a state highway system as a whole

Recommendation No 6 is deserving of particular notice as to the question of policy in connection with the supply of materials essential to roadside improvement work The business of properly designing and executing the highway projects is a big job in itself, and deserves the fullest attention of the highway organization if it is to be well done The business of producing and propagating material for planting use is the specialty of nursery organizations experienced and equipped for such work The committee's recommendation indicates that careful consideration was given to the possible sources of supply for the materials needed, which tends to show that plant requirements may be purchased satisfactorily on the basis of definite standard specifications in a similar manner as other highway materials

In the highway program, roadside development should be organized and handled on the same basis as the regular older types of highway work Road materials such as cement, stone, asphalt, and steel, are usually contracted for under competitive conditions Should not plant materials be secured in the same way, wherever and whenever required? Fair competition under carefully prepared specifications should insure reasonable prices for average needs While it is quite possible that in some localities or regions, where suitable nursery material may not be available in sufficient quantities, resort may have to be made to the use of some collected stock growing in the vicinity of the work,—still it is well to bear in mind that as time goes on it is reasonable to expect that opportunities of this kind are necessarily limited and may gradually dry up as a dependable source of supply The last recommendation is particularly important, since present indications point to the expansion of roadside improvement operations into quantity production over a reasonably extensive mileage

Conservation policies should be kept in mind throughout any highway program The collecting grounds of woods and mountains cannot be always depended upon as a constant supply of plant needs Robbing Peter to pay Paul is not the most satisfactory way to get plant materials for use along the roadsides, except for emergency use and in limited quantity The last recommendation of the Committee's report on policy therefore, is to be commended as showing the practical vision of the committee in looking forward to a reasonably continuous and relatively permanent program of roadside development that aims to insure an economic source of supply of plant material for anticipated requirements

MR WALTER D LUDWIG, *Pennsylvania Highway Department* In Pennsylvania we have been carrying forward about the same type of work that Mr Keith has so well started in Connecticut although we have some drawbacks For instance we do not own all of the trees within the right-of-way We only own those trees which are less than four inches in diameter at 2½ feet above the ground All others belong to the abutting property owner so it is a little difficult to do many of the things which Mr Keith reported However, this is a situation we must consider, in fact, all of the highway departments will likely have to consider it very shortly, if they have not already done so, or have proper laws to take care of it

I believe that most engineers will agree that a road is not finished until the whole road is completed including the area outside the ditch line. There was a time when all the engineer was interested in was the building of a nice piece of concrete or a nice piece of macadam, smooth, with good alignment and easy curves and grades, but most engineers have gotten over that idea today, and I think I am safe in saying that a majority of them agree that the road is not completed until the roadsides are cleaned up and made presentable

What constitutes most of the traffic along a highway? Is it purely commercial or is it largely business or business combined with pleasure? Perhaps some of you men came to Washington in your automobile—you came in a car and you came on business At the same time you combined business with pleasure and when we look at it in that light we might say that perhaps from 75 to 85 per cent of traffic on our highways sees and looks at the roadside, so that it is time that we look at the roadside as a definite part of the road

When you remember that you can sell the idea of roadside development on the utilitarian and safety features, such as, the opening up of curves and the treatment of slopes with some grass seed or perhaps a few flowers, taking care of clear-sight vision for traffic, and preventing and controlling erosion of the slopes, you certainly will have no trouble in making progress

MR M W TORKELSON, *Wisconsin Highway Commission* The problem with respect to roadside improvement is not so much that of deciding what is to be done as it is to coordinate the roadside development work with the regular construction and maintenance operations It has been my theory that the foundation for all roadside development is the original grading of the highway—that this is really what makes or mars a road The broad road beds, the gentle slopes, the shallow ditches have been adopted quite largely as standard practice with us The next step for us to take is to follow out the recommendation made by the committee, the value of which is self-evident, namely, to conserve the rich soil and to use it to cover the shoulders and slopes where we wish grass to grow The fine rich black soil is just as much a detriment to the stability of that portion of the road bed which supports the road as it is a benefit to the surfaces of the shoulders and the back slopes where we wish to grow a good thick grass for the beauty of the roadside and for the prevention of erosion We would serve ourselves two ways by getting this dirt out of the road bed and on to the shoulders

REPORT OF COMMITTEE ON MATERIALS AND CONSTRUCTION

H S MATTIMORE, *Chairman*

Engineer of Tests and Materials Investigations, Pennsylvania Highway Department

LIQUID BITUMINOUS MATERIALS FOR PLANT MIXED SURFACES

BY PREVOST HUBBARD

Chemical Engineer, The Asphalt Institute

AND C S REEVE

The Barrett Company

SYNOPSIS

The characteristics of petroleum distillates, rapid curing asphalt cut-backs, medium curing asphalt cut-backs, and slow curing petroleum residuals or blends, in relation to their use as primers and binders for aggregates are discussed These, with asphalt emulsions, are the general classes of liquid petroleums and asphaltic products commonly used in the construction of cold-laid plant mixed surfaces Recently developed specifications for such products are given in Table I

The liquid tar products adapted to various types of plant mixes are described and their ranges in characteristics are illustrated by a tabulation (Table II) of their consistencies, with references to suitable specifications Table III shows the types and ranges in percentage of tar binders for use with representative gradings of aggregates

PETROLEUM AND ASPHALTIC PRODUCTS

There are five general classes of liquid petroleum and asphaltic products commonly used in the construction of cold laid plant mixed surfaces

1 Petroleum distillates (naphthas)
2 Rapid curing cut-backs (asphalt cement and rapidly volatile distillate)
3 Medium curing cut-backs (asphalt cement and medium volatile distillate)
4 Slow curing petroleum residuals or blends
5 Asphalt emulsions

As asphalt emulsions have been made the subject of a separate report only the first four will be discussed in this paper

Two main divisions of the liquid products may be made according to the purpose they are intended to serve and these may be conveniently subdivided as shown in the following classification

I Primers for Aggregates
- a Petroleum distillates (commonly termed liquefiers) the use of which is followed by application of a coating of asphalt cement
- b Medium curing cut-backs of low viscosity that produce a very thin coating of asphalt upon which a thicker film of asphalt binder may be readily deposited

II Binders for Aggregate
- a For open-graded aggregates, containing little or no 200 mesh particles
  - 1 Rapid curing cut-back asphalts for commercial crushed aggregate products
  - 2 Medium curing cut-back asphalts for gravel and sand aggregates in which there exists a regular gradation in size down to and including particles passing 10 mesh
- b For dense graded aggregates containing over 5 per cent of 200 mesh particles
  - 1 Medium curing cut-back asphalts
  - 2 Slow curing products

PRIMERS

Distillate primers are employed not only for the purpose of facilitating the uniform coating of a cold aggregate with hot asphalt cement but of temporarily softening the film of asphalt surrounding each aggregate particle enough to permit ready workability and compressibility of the mixture during construction Petroleum naphtha or gasoline is ordinarily used Specifications for liquefier have not been standardized but typical requirements call for a product showing an end point on distillation of not over 450 or 500°F with 45 to 50 per cent distilling at not over 293 to 325°F While some engineers lay considerable stress upon the particular grade of naphtha and require a heavier less volatile grade for delayed use of the mixture, as compared with immediate use, fine distinctions along this line are of doubtful importance as the proportion of distillate added to the mix with relation to the proportion of asphalt cement and absorptive character of the particular aggregate can be made to produce greater differences in the so-called setting qualities of the mixture than will be produced by slight differences in distillation ranges of the liquefier.

Medium curing cut-backs have not as yet been extensively used as aggregate primers but they possess certain advantages over the distillates which are of considerable practical value In the first place they immediately produce a thin tenacious moisture resistant coating of asphalt over each particle which serves as an adequate base for uniformly applying a thicker film of either an asphalt cement or a highly viscous cut-back asphalt Moreover, once the aggregate has been coated with the thin film of asphalt it may be stock piled indefinitely with practically no adhesion between the individual particles and may be later cold mixed with the asphaltic binder immediately prior to construction It is of course important that the medium curing primer be of very low viscosity so that a small percentage may be rapidly mixed with the aggregate to produce thin films of asphalt cement Typical requirements for such a product are shown in the accompanying table of specifications

BINDERS FOR OPEN GRADED AGGREGATES

When the inherent stability of an aggregate is entirely dependent upon the intimate interlocking of its individual particles, as typified by a single commercial size of crushed stone, it is essential that the asphaltic product with which it is to be mixed should possess or quickly develop a strong mechanical bond to resist successfully displacement of the mixture under traffic Moreover the coating or film of binder on each individual particle should be as heavy as consistent with good workability in order to produce a bond which is durable and which, if disturbed, will readily reform A rapid curing cut-back asphalt of as high an initial viscosity as is practicable to incorporate with the cold aggregate is best adapted for this purpose, and in general, the coarser the aggregate the higher should be the viscosity of the cut-back in order to secure the most satisfactory immediate results A Furol viscosity as high as 700 to 1400 at 140°F is desirable when all of the aggregate is retained on the ¼ inch screen and the maximum size runs as high as 1½ inches This is particularly true when the mixture is to be laid immediately after preparation during warm weather When it is to be stored or transported for long distances before use or when it is to be used for cold patching or to be laid in cool weather, the viscosity of the binder may, however, be lowered advantageously

When the inherent stability of the aggregate is largely dependent upon the presence of relatively small diameter particles which pack the voids between the larger particles, as typified by a gravel containing a substantial proportion of sand but free from material passing the 200 mesh sieve, the choice of the most suitable liquid asphaltic binder becomes a matter of experience and judgment which is difficult to reduce to hard and fast rules Choice may lie between a rapid curing cut-back of relatively low viscosity or a medium curing cut-back of higher viscosity

When the percentage passing the 10 mesh sieve is insufficient to pack closely the voids between the coarser particles a rapid curing product may often be used to advantage In other cases, except for sand aggregates, the medium curing type is to be preferred Even with the sand aggregate it may often be advisable to add 200 mesh material and use a medium curing cut-back In general the medium curing product is preferable except for base course construction rather than the use of unaltered sand with a rapid curing product

BINDERS FOR DENSE GRADED AGGREGATES

Dense graded aggregates are those with a more or less continuous gradation in size from the maximum diameter particle down to and including a substantial percentage of mineral matter passing the 200 mesh sieve Specifications for such aggregates commonly call for 7 to 14 per cent passing the 200 mesh sieve Many aggregates of this type, as represented by pit run gravel and crusher run stone, produce excellent wearing courses without any bituminous binder when a certain optimum water content is present Strength of bond of the bituminous binder with which they may be mixed is therefore not as important as in the case of open graded aggregates However the viscosity of the binder should be high enough to permit a sufficient quantity being used to prevent absorption of large amounts of moisture and displacement of the bituminous films by water

Rapid curing cut-backs are usually unsatisfactory for use with the dense graded aggregates because they do not distribute uniformly but tend to ball in the presence of appreciable quantities of 200 mesh particles The choice therefore lies between a medium curing cut-back and a slow curing product There is little to be said in favor of the latter except its usually lower cost Where traffic and moisture conditions are not severe the slow curing product has in many instances produced excellent low cost road mixtures but generally speaking, use of a medium curing cut-back is a much safer proposition In either case the viscosity of the product should be as high as possible without interfering with the workability and compressibility of the mixture During and after construction the medium curing cut-back increases in viscosity to a much greater extent than the slow curing product, which is a distinct advantage

CHARACTERISTICS OF LIQUID ASPHALTIC PRODUCTS

During the past three years as the result of extensive cooperative work between the United States Bureau of Public Roads, the various state highway departments and the producers of asphalt, a fairly complete set of specification requirements has been developed for liquid asphaltic road materials Such of these specifications as are applicable to cold laid plant mix construction are shown in Table I to illustrate the preceding brief discussion The development of the medium curing cut-backs shown in this table has been of rather recent origin and many highway engineers have not as yet become acquainted with their pecul-

TABLE I

LIQUID ASPHALTIC MATERIALS FOR PLANT MIXED SURFACES

Use	Primer	Binder						
Aggregate	Crushed Aggregate	Open Graded, containing little or no 200-mesh: Commercial Crushed Aggregate			Open Graded, containing little or no 200-mesh: Gravel and Sand		Dense Graded, containing 7–14% of 200 mesh	
Type Asphaltic Product	Medium Curing	Rapid Curing			Medium Curing		Medium Curing	Slow Curing
Water and Sediment	—	Free-from	Free-from	Free-from	Free-from	Free-from	Free-from	2–
Flash Point (Cleveland Open Cup) °F	—	—	—	—	—	—	—	250+
Flash Point (Open Tag) °F	—	80+	80+	80+	150+	150+	150+	—
Furol Viscosity at 77°F	40–150	—	—	—	—	—	—	—
Furol Viscosity at 122°F	—	200–400	—	—	—	—	—	—
Furol Viscosity at 140°F	—	—	275–400	700–1400	500–800	—	300–500	350–550
Furol Viscosity at 180°F	—	—	—	—	—	170–280	—	—
Distillation, per cent by volume								
Total distillate to 437°F	10–	10+	3+	0.5+	1–	1–	2–	2–
Total distillate to 600°F	25+	20+	14+	7+	16–	14–	8–20	8–
Total distillate to 680°F	50–	35–	30–	25–	25–	20–	25–	18–
Tests on Residue from distillation								
Float at 122°F	—	—	—	—	—	—	—	25+
Penetration at 77°F, 100 g, 5 sec	70–300	60–120	60–120	60–120	100–300	100–300	100–300	—
Ductility at 77°F	60+	60+	60+	60+	60+	60+	60+	—
Per cent soluble in Carbon Disulphide	99.5+	99.5+	99.5+	99.5+	99.5+	99.5+	99.5+	99.0+

TABLE II

CONSISTENCIES OF TYPICAL TAR PRODUCTS

	A	B	C	D	E
Specific Viscosity at 40°C	8–13	—	—	—	35–80
Specific Viscosity at 50°C	—	26–36	—	—	—
Float Test at 50°C	—	—	40–80	60–120	—
A S T M Specifications	D-104-30 D-105-30	D-104-30 D-105-30	D-110-30 D-111-30	D-110-30 D-111-30	D-106-28 T D-107-28 T

A is a thin fluid product, commonly used for surface treatment but also suitable for a primer to be used on aggregate in place of the more frequently used volatile solvent

B is a fairly viscous refined tar product adapted for use in road mixtures as distinguished from plant set-ups, particularly with graded broken stone and gravels which will furnish a well-graded stable mixture, not too dense to permit adequate setting-up within a short period Such a mixture, for example, may correspond to the "retread" type of surfacing, but may include aggregates of smaller maximum size than are commonly used for "retread" work

C is a viscous refined tar especially adapted for use with graded fine aggregate Due to its greater initial adhesiveness, it may be used with aggregates of closer grading than material "B," and ordinarily in such mixtures a certain proportion of fine aggregate is desirable Mixtures with this grade of tar product may be made ordinarily with aggregate at summer atmospheric temperature, and may be laid without heating The tar must be heated before mixing with the aggregate While it is especially adapted for use with graded fine aggregate, it may also be used in road mixed work where a heavier binder than "B" is required

D is a grade of refined tar heavier than "C" and almost semi-solid, requiring its heating before use with aggregate It is adapted for use with graded coarse aggregates, or at summer temperatures with fine aggregate such as described for material "C" Such mixtures with material "D" may also be made ordinarily with aggregates at summer atmospheric temperatures and may be laid without heating

"E" is a cut-back tar product commonly used in patching mixtures but also well adapted for use with graded aggregates of maximum sizes ranging from ¼ inch to 1½ inches Material of this viscosity may be used at ordinary temperatures, and only in cool weather is slight heating necessary or desirable

TABLE III

Percentage Passing	Fine	Intermediate	Coarse	Medium Patching	Fine Patching
2¼ Inch Screen	—	—	100	—	—
1½ Inch Screen	—	100	—	—	—
1¼ Inch Screen	—	—	30–60	—	—
1 Inch Screen	—	—	—	100	—
¾ Inch Screen	—	30–60	—	—	—
⅝ Inch Screen	100	—	—	—	100
½ Inch Screen	—	—	—	35–70	—
¼ Inch Screen	40–70	0–5	0–5	10–25	40–70
No 10 Screen	10–35	—	—	—	10–35
Percentage of Tar Binder	7–9	3–5	2 5–4 0	5–7	7–9
Type of Tar Binder	C or D*	D	D	E	E

* Selection dependent on season, method of handling, etc

iar advantages for use with certain types of aggregate. They are particularly well adapted for utilizing pit run gravel and crusher run stone when the aggregate has any merit as a road building material.

TAR PRODUCTS

The range of liquid tar products adapted to various types of plant mixes is best shown by consistencies of typical tar products in Table II The other characteristics will substantially conform to requirements of the A S T M specifications indicated under each type, except as distillation limitations are necessarily modified by consistency Considering the wide variation in aggregates available for plant mixtures and in service conditions, products of other consistencies than shown are adapted for use, it is advisable, however, from a manufacturing standpoint to limit the number of grades of material, provided that a sufficient number are provided for satisfactory utilization under all important conditions

While there is a wide variation in preferences for grading of aggregates in plant mixes, the gradings shown in Table III are fairly representative of average practice The type and proper range of percentage of tar binder is also shown

PROGRESS REPORT ON THE REACTION OF CALCIUM CHLORIDE ON PORTLAND CEMENT[1]

By Paul Rapp, *Research Associate at the Bureau of Standards for the Calcium Chloride Association*, and Lansing S Wells, *National Bureau of Standards*

SYNOPSIS

The addition of calcium chloride appears to increase somewhat the heat contributed at the end of 24 hours by dicalcium silicate and tetracalcium ferro-aluminate and to decrease the heat from tricalcium aluminate present in cements The heat contributed by tricalcium silicate shows very little change when calcium chloride is added Calcium chloride increases the rate at which the heat is evolved from all cements tested and in general gives an increase of about four calories per gram of cement at 24 hours It decreased the time of set of the 11 commercial cements and increased the flow of the concrete mix and the strength of the resulting concrete at all ages to 90 days, beyond which results have not yet been obtained

INTRODUCTION

The Calcium Chloride Association has maintained a fellowship at the Bureau of Standards to study the reactions of calcium chloride with cements and their constituents and to obtain more information on the effect of this material on concrete made from present day cements The results of the complete investigation of the effects of calcium chloride on cements and concrete cannot be given at this time since the long-time tests have not been completed However, it is felt that certain salient factors are of sufficient interest to those concerned with the use of calcium chloride in concrete to warrant a brief paper at this time on some of the results obtained Therefore the data herewith presented relate to the short-time tests and include the effect of calcium chloride on the heat of hydration, setting time, strength and consistency of a selected group of cements

It is planned to publish that phase of the study dealing with the physico-chemical reactions between calcium chloride and the constituents of cement as a separate paper

DESCRIPTION OF THE CEMENTS AND CALCIUM CHLORIDE

Eight portland cements (referred to in this report as standard cements) together with one high-early strength and two white portland cements were studied in this investigation In selecting these cements an effort was made to obtain the greatest variation possible in composition and physical properties

[1] Publication Approved by the Director of the Bureau of Standards of the U S Department of Commerce

Sixty experimental cements of varying composition were furnished by the Portland Cement Association Fellowship at the Bureau of Standards for certain heat studies herein reported

The calcium chloride used was a commercial flaked hydrated product containing 77 1 per cent $CaCl_2$ and the amounts added to the cements are reported as percentage $CaCl_2$ by weight of cement.

EFFECT OF CALCIUM CHLORIDE ON HEAT EVOLVED DURING FIRST TWENTY-FOUR HOURS OF HYDRATION

General Consideration. It has long been recognized that certain reactions which take place during the setting and hardening of cements result in an evolution of heat. During the past few years there has been an added interest in the quantity of this heat evolved as it relates to the properties of the concrete. It seemed advisable, therefore, to determine the effect of the addition of calcium chloride upon the heat developed as cement hydrates

From measurements of the temperature rise of a given mixture of cement and water during the first twenty-four hours the quantity of heat evolved during this period can be calculated This method also gives the rate at which the heat is evolved

Heat of Hydration of Sixty Special Cements The sixty samples of laboratory cements above referred to had a wide range in composition which made them of especial value in the present study. The analytical data and heats of hydration in the absence of calcium chloride were kindly furnished by the Portland Cement Association Fellowship The quantity of each cement available was so limited that only one additional calorimetric determination (using one per cent anhydrous calcium chloride) could be made.

The apparatus used for measuring directly the heat of hydration was that developed at the Bureau of Standards.[2] The cement (200 grams) and water (67 5 grams) were thoroughly mixed in a small tinned can by a high speed mixing device Copper-constantin thermocouples were inserted into the mixture and the can placed in a double-walled vacuum flask which was in turn tightly closed and placed in an air thermostat maintained at 21 $\pm 1°C$. The apparatus was so arranged that eight separate determinations could be made simultaneously

From the data obtained in this manner it was possible to calculate the heat evolved by the cement at any given period[3] after making the necessary correction for the radiation loss of the vacuum flask and its contents

The contributions of the individual compounds to the heat of hardening were calculated by the method of least squares after computing the

[2] Variations in Standard Portland Cements, P H Bates, Jour Amer Concrete Institute, Vol 1, page 65, Nov 1929

[3] The Heat of Hydration of Portland Cement Paste, Lerch and Bogue, Bureau of Standards Jour of Research (RP684) Vol 12, pp 645–664, May, 1934

percentages of the compounds in the cements by the method of Bogue[4] and assuming that a linear relationship exists between the compound composition of the cement and the heat evolved The values obtained from these cements (Table I) show the same trends as those reported by Blank[5] and by Wood and his co-workers,[6] namely that tricalcium aluminate $3CaO\ Al_2O_3$, gives the greatest heat evolved (expressed in calories

TABLE I

CONTRIBUTION OF INDIVIDUAL COMPOUNDS TO HEAT OF HARDENING
(In calories of each per cent in one gram of cement)

Compound	Value for 0% $CaCl_2$ at 1 Day	Probable Error	Value for 1% $CaCl_2$ at 1 Day	Probable Error	Blank's Value at 1 Day	Wood s Value at 3 Days
Tricalcium Silicate	80	± 02	78	± 02	57	98
Beta Dicalcium Silicate	19	± 02	26	± 02	095	195
Tricalcium Aluminate	1 62	± 07	1 47	± 06	2 00	1 70
Tetracalcium Ferro-Aluminate	01	± 08	25	± 06	19	29

TABLE II

CHANGES IN THE TOTAL HEAT EVOLVED AT TWENTY-FOUR HOURS BY ADDITION OF ONE PER CENT OF CALCIUM CHLORIDE TO EXPERIMENTAL CEMENTS

Grouping—Heat Evolved 24 Hours No $CaCl_2$	No of Cements in each Group	Average Changes Produced by 1% $CaCl_2$	Average Compound Compositions			
			C_3A (1)	C_4AF (2)	C_2S (3)	C_3S (4)
(Calories per g)		*(Calories per g)*	%	%	%	%
30–39 9	9	+7	3	15	47	30
40–49 9	12	+5	4	14	35	42
50–59 9	24	+4	10	12	34	42
60–69 9	16	0	10	11	25	52
70–79 9	3	−2	15	7	21	53

(1) $C_3A = 3CaO\ Al_2O_3$.
(2) $C_4AF = 4CaO\ Al_2O_3\ Fe_2O_3$.
(3) $C_2S = 2CaO\ SiO_2$ (Beta)
(4) $C_3S = 3CaO\ SiO_2$.

for each per cent present in each gram of cement) tricalcium silicate, $3CaO\ SiO_2$, being next in order The addition of calcium chloride appears to increase somewhat the heat contributed by dicalcium silicate, $2CaO\ SiO_2$ and tetracalcium ferro-aluminate, $4CaO\ Al_2O_3\ Fe_2O_3$ and to decrease the heat from tricalcium aluminate The heat contributed

[4] Calculations of the Compounds in Portland Cement, R. H Bogue, Ind & Eng Chem , Anal Ed 1, page 192, 1929

[5] Comparison of Selected Portland Cements in Mass Concrete Tests, Robert T Blank, J Am Concrete Inst , Vol 5 (No 1), page 9, 1933

[6] Effect of Composition of Portland Cement on Heat Evolved During Hardening Wood, Steinam and Starke Ind Eng Chem , Vol 4, p 1207 (1930)

by tricalcium silicate shows very little change when calcium chloride is added

In Table II the experimental cements are divided into five groups on the basis of the calories evolved per gram of cement at the end of twenty-four hours without addition of calcium chloride Column 1 gives the grouping, Column 2 the number of cements in each group, Column 3 the average change in the heat developed in 24 hours produced by the addition of one per cent calcium chloride and the remaining four columns the average compound composition From this table it can be seen for the experimental cements that an increase in the calories of heat evolved from group to group was accompanied by an increase in percentage of both the tricalcium aluminate and tricalcium silicate, the two compounds as shown from Table I to be those contributing the greater quantities of heat of hydration Furthermore, the increase in the heat evolved because of the addition of one per cent calcium chloride is greatest in the first group (cements of lowest heat), becomes less in the succeeding groups, reaching zero for the fourth group, and in the fifth group there is an actual decrease of two calories These changes in heat evolved because of the addition of calcium chloride cannot apparently be assigned to the tricalcium silicate content in each group since it has been shown in Table I that each per cent of tricalcium silicate present in one gram of cement evolves the same number of calories with and without calcium chloride The changes do, however, appear to be inversely related to the amounts of tricalcium aluminate and directly to those of dicalcium silicates and tetracalcium ferro-aluminates present in each group and in this respect are in agreement with the data of Table I

Heat of Hydration of Eleven Commercial Cements The heat evolved from eleven commercial cements during the first 24 hours of hydration was measured with and without calcium chloride to determine the nature of the change produced by the addition of 0 5, 1, 1 5 and 2 per cent of anhydrous calcium chloride

The heat data pertaining to these commercial cements were analyzed in the same manner as described for the experimental cements and similar conclusions were obtained as to the contribution of heat by the individual compounds present as well as to the effect of calcium chloride

The effect of the addition of calcium chloride on the heat curves for cement B (Standard portland) is shown in Figure 1 As the amount of calcium chloride is increased the rate of heat evolution is increased The addition of 1 5 per cent of anhydrous calcium chloride though increases this rate proportionately more than two per cent The amount of heat developed at 24 hours does not change more than a few calories for the various percentages of calcium chloride added The other seven standard and the two white cements behave in a similar manner upon the addition of calcium chloride

Time-temperature curves plotted from the temperature data pertaining to these cements show that calcium chloride generally increases the rate of temperature rise and decreases the time to reach the maximum temperature. However, with the high-early strength cement the maximum temperature attained both with and without calcium chloride was at or near the boiling point of water so that the heat of hydration could not be ascertained satisfactorily since an undetermined portion of the heat was involved in the latent heat of vaporization of water.

EFFECT OF CALCIUM CHLORIDE ON THE PHYSICAL PROPERTIES OF MORTARS AND CONCRETES

In order to obtain a measure of the effect of calcium chloride on the physical properties of the cements in mortars and concretes it was deemed advisable to control all factors, as far as possible, so that the admixture of calcium chloride would be the main variable.

Making the Test Specimens. The usual evaluation tests for portland

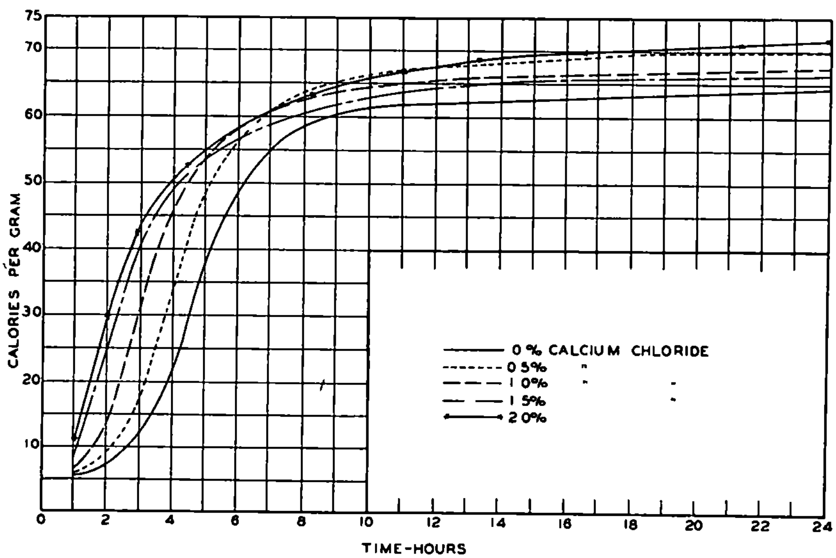

Figure 1. Heat Curves for Cement B

cement[7] were made except that two inch mortar cubes were substituted for the briquets. In these tests 0, 0.5, 1, 1.5 and 2 per cent anhydrous calcium chloride, respectively, were added to the gauging water. The mortar specimens were kept in the molds in a moist closet at 70°F. for the first 24 hours, removed and stored in water at the same temperature until tested.

Concrete was tested in the form of 6 x 12 inch cylinders made from a 1:2:4 mix by volume, using local sand and gravel. The amount of

[7] Federal Specification SS-C-191 for Cement, Portland; Standard Methods of Sampling and Testing Portland Cement. Am. Soc. Testing Materials, C77-32.

water (6 5 gallons per sack) used for gauging gave a cement-water ratio of 1 73 by weight and contained, respectively 0, 1, 1 5 and 2 25 per cent of anhydrous calcium chloride by weight of the cement

The effect of two curing temperatures during the first 24 hours was studied by means of plastic mortar tests [8] Pit run Ottawa sand was used as the aggregate The same cement-water ratio and percentages of admixtures of calcium chloride were used as in the concrete tests One group of specimens was made and stored at 70°F until tested Another group was molded at 90°F and stored at that temperature for the first 24 hours The specimens were then removed from the molds and placed in water at 70°F.

Results The times of set are shown in Figure 2 It will be noted

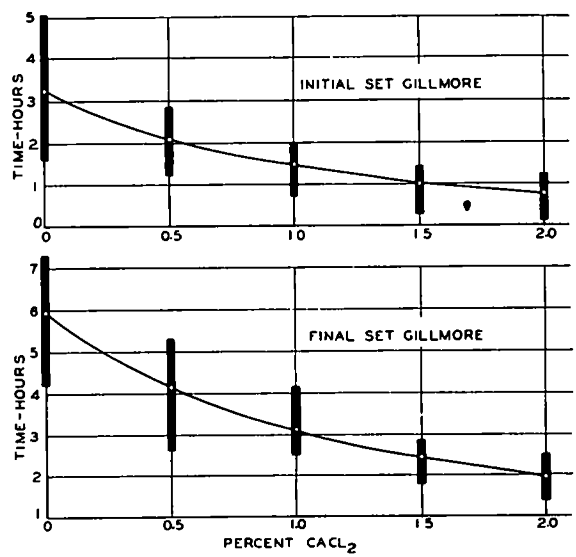

Figure 2 Setting Time of Standard Portland Cements

that the average setting time, together with, in general, the spread of the time of set decreases as the amount of calcium chloride is increased Similarly the addition of calcium chloride to the white and high-early strength cements, not given in Figure 2, shows the same effect as obseved with the standard cements All pats, both with and without calcium chloride, were sound

A summary of the concrete strengths obtained on eight standard cements is given in Figure 3 The concretes with all percentages of calcium chloride show increased strength over the plain concrete at all ages tested At one day the average strength of the concrete with 1 5 per cent admixture was 123 per cent higher than the average strength of

[8] A Plastic Mortar Compression Test for Cement E M. Brickett, Am Soc Testing Mats Vol 28, (Part II) page 43, 1928

the plain concrete. At 28 days the increase was 13 per cent and at 90 days 9 per cent, indicating that calcium chloride has a greater accelerating action at early ages Figure 3 indicates that there is very little advantage of adding more than two per cent commercial (1 5 per cent anhydrous) calcium chloride, the amount generally recommended for use at 70°F. The strength of the concrete containing white cement was within the same range as for those with standard cements and calcium chloride had apparently the same effect thereon The strength of the concrete made with high early strength cement was also increased by the addition of calcium chloride However, the strengths were higher than those of the eight standard cements in all cases

The effects of the addition of calcium chloride on the strengths of the

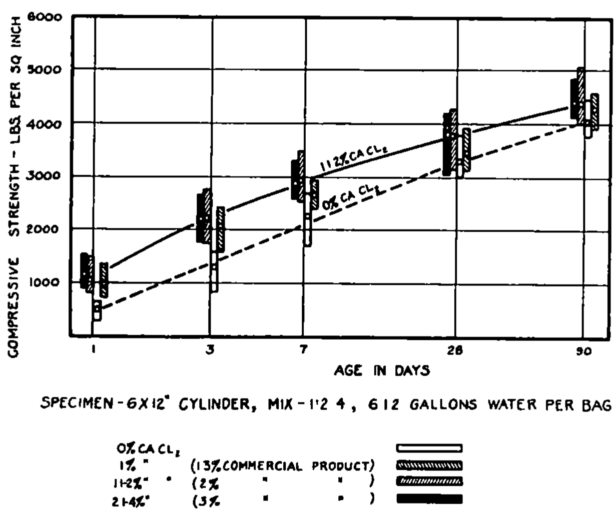

Figure 3. Concrete Strength Summary of Eight Standard Portland Cements

cements when tested in 1:3 standard sand mortars showed the same trends as observed on the concrete strengths

The tests of the strength at different curing temperatures have been undertaken to ascertain how strength of concrete may be affected by the addition of calcium chloride under the temperature conditions encountered during the construction season Specimens have been made for two curing temperatures (70 and 90°F) and curing a group of specimens at a temperature of 40°F is being studied Since this part of the investigation is not finished a tabulation of the results obtained to date will not be included in this report It is of interest, however, to note here that the one day specimens without calcium chloride cured at 90°F. have approximately the same strength as the 70°F specimens containing calcium chloride

During the molding of both the mortar and concrete specimens a greater ease of placing was noted when calcium chloride was incorporated in the mix To obtain some measure of this phenomenon, flow measurements were made on the concrete With each cement flows were increased by additions of calcium chloride to the concrete The range of flows obtained on eight standard cements as well as the average flow are plotted in Figure 4 against the amount of calcium chloride added The flow is expressed as the per cent increase in diameter after dropping the flow table 15 times, a distance of ⅛ inch

Each point on the average flow curve represents the mean of 72 determinations There was an increase in the average flow from 29 to 41 with the addition of 1 5 per cent anhydrous calcium chloride. The increase in flow was more marked in the range of 0 to 1 5 per cent calcium chloride than beyond 1 5 per cent.

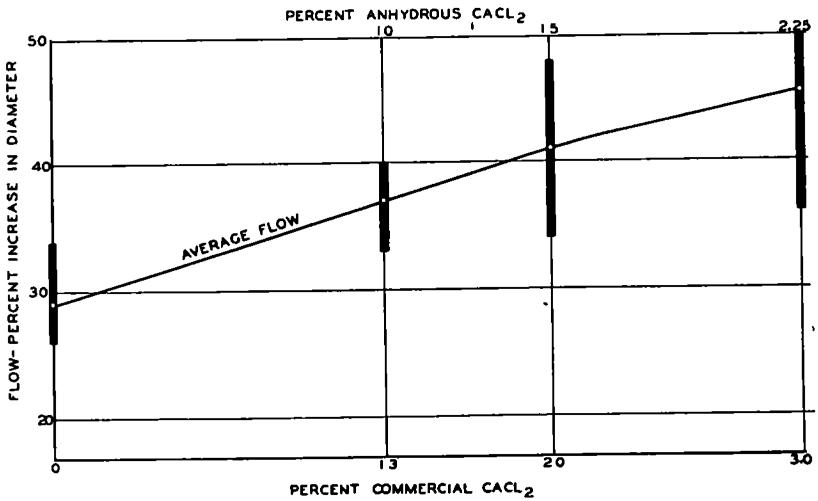

Figure 4 Flow Tests of the Concrete of Eight Standard Portland Cements Mix—1 2.4. 6 5 gallons water per bag

The flows of the concrete containing white cement were within the same range as for those with standard cement, and the calcium chloride had approximately the same effect The flow of the concrete containing high-early strength cement was similarly increased by the addition of calcium chloride but the flow was smaller in all cases, being 16 with 0 per cent and 21 with 1 5 per cent calcium chloride.

SUMMARY

In considering the results obtained in this investigation, it should be borne in mind that the tests are by no means finished, and that later information may alter some of the following tentative conclusions

1 Calcium chloride increases the rate of heat evolution of all cements included in the investigation

2 Calcium chloride increases the total heat of hydration of low heat

cements and may decrease the heat of hydration of high heat cements at 24 hours

3 The chemical composition of the cement has an important effect upon both the rate and total amount of heat of hydration developed during the first 24 hours The general effect of calcium chloride over the range in compositions studied was a tendency to level the heat of hydration to an average increase of about four calories per gram at 24 hours

4 The times of set of 11 commercial cements were decreased by adding increasing amounts of calcium chloride The greatest proportional decrease was obtained by the addition of two per cent commercial calcium chloride

5 Additions of calcium chloride did not affect the soundness of the cements

6 The addition of calcium chloride increased the strength of the concrete made from the cements tested at all ages to 90 days, beyond which results have not yet been obtained

7 The flow of concrete was increased by the addition of calcium chloride

ACKNOWLEDGMENT

In carrying out the work in connection with this paper the authors wish to thank Chester A Hogentogler, Jr , for his assistance in preparing many of the test specimens and Wilder D Foster in calculating much of the heat data Thanks are also expressed to P H Bates and H F. Clemmer for their assistance in the progress of this research

REPORT OF PROJECT COMMITTEE ON FILLERS AND CUSHION COURSES FOR BRICK AND BLOCK PAVEMENTS

JOHN S CRANDELL, *Chairman*

Professor of Highway Engineering University of Illinois

SYNOPSIS

The report describes the recent use of a number of new fillers for brick pavements Among them are Temperature Resisting Cement, a bituminous compound known as Zorbit, an asphalt-rubber compound and Trinidad asphalt Means for removing filler from the top surface of the brick pavement are described To facilitate the removal, the brick have been coated with various materials, among them being whitewash, calcium chloride, and several proprietary materials All are used successfully with proper precautions and methods

Several new materials have been advocated for fillers During 1932 Temperature Resisting Cement was used on two jobs in Ohio, and from these it was learned that: (a) the sand used with the cement should have a greater percentage of fines than is ordinarily found in concrete sands, (b) because of water repellancy of the cement, caused by the bituminous film thereon, a longer time and a more thorough mixing is requisite, and (c) it is not good practice to dump the filler direct from the mortar box to the pavement as there is likely to be some separation of ingredients

Experiments are being conducted with a bituminous compound known as Zorbit This is a powdered asphalt which is mixed with a flux of such character that the resultant mixture is fluid for a reasonable length of time, sets with a minimum of shrinkage It adheres well, and seems to be capable of holding grit or other skid-proofing substances

Mention was made in the 1932 Proceedings of an asphalt-rubber compound which has been developed at the University of Illinois This has been wholly satisfactory in a small experimental patch in a pavement at Urbana, Illinois It did not run in summer, and it is not brittle in cold weather It's one disadvantage is that is does not flow readily enough to enter the joints of a lugless brick pavement, although it does fill the joints of a lug brick job

Trinidad asphalt filler was used experimentally on January 6, 1933 at New Castle, Pennsylvania on U S Route 422, between Chestnut and Walnut Streets An area of 383 square yards was filled, using 26 pounds of asphalt per square yard The brick were laid with wide joints, to correspond with the old pavement already in place, on a sand cushion The asphalt was pushed into the joints with the customary squeegee, and a sand cover was applied at 10 lbs per square yard

REMOVAL OF FILLER FROM TOP OF PAVEMENT

Numerous attempts have been made to find a material that may be applied to the surface of the bricks so that the bituminous filler may be peeled off without removing it from the joints themselves Whitewash, calcium chloride, and proprietary "separators" have been used successfully At first it was thought that whitewash should be applied so as to leave a heavy coat, and that this should be dry when the bitumen is applied however it has been found that if the filler is immediately peeled off damp whitewash is better than dry. If calcium chloride solution is used the filler may be applied as soon as the bricks are sprayed, and the excess should be quickly scraped or peeled off Going one step further, it is probable that in moderate weather it may be possible to do a good job by merely sprinkling the top of the pavement with water alone So long as there is a film of moisture present to prevent the asphalt from adhering, there will be little difficulty in removing the film. In hot weather water alone would evaporate too quickly and calcium chloride is indicated One very important item to watch is that of foaming If too much watery solution is present then the filler will foam to such an extent that the joints will not be filled And furthermore, since the material recovered is put back in the heating kettle, there must be a minimum of water present in it or there will be foaming in the kettle, which leads to a fire

The 1933 specifications for Ohio do not permit of the filler being squeegeed into the joints. It is to be poured over the surface or into the joints, and then the excess removed It has been found in practice that thin films of bitumen are difficult to scrape off, whereas thick films or layers are easy to remove in any weather

The Committee will investigate various types of cement grout filler during the coming year.

DISCUSSION

ON

FILLERS AND CUSHION COURSES FOR BRICK AND BLOCK PAVEMENTS

Mr Geo F Schlesinger, *National Paving Brick Association* The part of the report which is very interesting to me is Professor Crandell's investigation of a new type of filler—some sort of rubber compound If we develop a filler that will not come up after it is in the joints there will be no danger of any subsequent effect which would serve to decrease the coefficient of friction. If Professor Crandell will discover some filler of that kind he will make a valuable contribution With a vertical fiber wire-cut lug brick we can use a much harder asphalt for filler than formerly, one that is not so susceptible to temperature changes Professor Crandell in his report also discussed the surface removal method which has now been made standard practice in a number of states where brick pavements are used—Ohio, Illinois, Michigan, Indiana, and I think Pennsylvania There have been enough jobs built to leave no question in my mind as to the merits of the surface removal method of filler application All engineers that have used it say that is the preferred method of applying bituminous material to fill brick pavements

MR H S MATTIMORE, *Pennsylvania Highway Department* We are rather interested in the methods used for removing the excess asphalt Is there any danger of those liquid solutions going into the joints of the brick and defeating the purpose of the bond?

PROFESSOR CRANDELL: Yes The workmen must be careful not to splash this material over the top Use a spray or a mist You need only the least bit of any of these materials to prevent adhesion provided you work quickly to remove the asphalt If you wait an hour or two you will find that water or calcium chloride may not work so well You will notice in the report in 1931 that a heavy coat of whitewash was recommended This was advocated so that you could go back week after next and still be able to remove the asphalt, because there is a layer of whitewash on the brick which does not permit the asphalt to adhere But this is not true if water or calcium chloride are used

MR SCHLESINGER: As far as foaming into the joints is concerned, there hasn't been any particular trouble It should be sprayed on and the surface kept just moist As between calcium chloride and whitewash, we recommend calcium chloride With whitewash the filler can be scraped off and it doesn't stain the surface of the pavement Nevertheless you get a good bright looking pavement with calcium chloride

A V. BRATT: How is the whitewash applied?

PROFESSOR CRANDELL: I prefer the spray equipped with a large nozzle Mr Mattimore brought up the question as to whether the calcium chloride would cause foaming in the kettle, and I have been fearful of that for some time A small amount of water in the kettle is dangerous

WEATHERING TESTS ON CONCRETE

By L O Hanson

Instructor in Mechanics, University of Wisconsin

SYNOPSIS

The paper presents results of tests to determine the effect of amount of mixing water and type of course aggregate on the durability of concrete The concretes were designed with three water-cement ratios, 0 65, 0 85, and 1 10 by volume, each with two slumps of 2 and 7 inches Four widely differing crushed limestones were used as course aggregates Alternate freezing and thawing was used as a weathering test both on the concrete and on samples of crushed limestone In addition, the crushed rock was given 5 and 10 cycles of a slightly modified A S T M soundness test with sodium sulfate The specimens of concrete were 4 by 6 by 18 inch beams and 6 by 12 inch cylinders

Indications from the tests are, in part· Weathering resistance of the concrete decreased with increase in water-cement ratio and increased with additional length of curing period The quality of the course aggregate was directly reflected in the resistance to freezing and thawing of the concrete Freezing and thawing tests and the sodium sulfate test on the limestones graded the rocks in the same relative order of durability, 25 and 100 cycles of freezing and thawing being roughly duplicated respectively by 5 and 10 cycles of the sodium sulfate test

This paper presents the results of certain tests made at the Materials Laboratory of the University of Wisconsin to determine the effect of amount of mixing water and type of coarse aggregate on the durability of concrete Four widely differing crushed limestones produced within the state were selected as coarse aggregates Alternate freezing and thawing was used as a weathering test both on the concrete and on samples of the crushed limestone, and, in addition, the crushed rock was subjected to five and ten cycle runs of a slightly modified form of the American Society for Testing Materials Tentative Method of Test for Soundness of Coarse Aggregate (C 89–32 T) This work, which is part of a more extensive series of weathering tests, has been carried out by the writer under the general supervision of Professor M O Withey, of the Department of Mechanics, and with the cooperation of the Wisconsin Highway Department, which furnished a portion of the necessary funds and aided in the securing of the rock samples

MATERIALS

The three principal limestones which underlie a large part of southern Wisconsin are· (1) The Lower Magnesian limestone (Ordovican) which is somewhat irregular in character, frequently cherty, variable in grain size and rather sandy, especially in the lower beds; it is hard and wear resistant and breaks with a hackly surface into blocks of irregular outline· (2) The Trenton (Ordovican) which is a thin bedded argillaceous magnesian limestone, parting planes with some clay often being very close together, it is relatively low in hardness and wear resistance (3) The Niagara (Silurian) which is a dolomitic limestone of fine texture, uniform quality, and is generally in massive beds, it is hard, has a low percentage of wear and breaks into roughly rectangular blocks of uneven dimensions

The Lower Magnesian is represented by one sample from quarry "C," a large roadside pit which has been used for a number of years Rock faces at this quarry are not badly weathered The Trenton is represented by quarry "A" which was a local pit just opened and in production at the time of sampling The material there was mostly thin bedded and rock faces which had been exposed for many years showed a considerable amount of disintegration "B" is a large commercial quarry in continuous operation in a Trenton formation which is more thickly bedded, harder and less badly weathered than is usual for the Trenton Quarry "D" in the Niagara is a large commercial quarry in continuous operation. The rock there is quite massive, of good texture and uniformity and shows little or no evidences of weathering on faces exposed for many years

Approximately a ton of fresh clean rock was gathered by hand at each quarry At the laboratory this was crushed to a maximum size of 1½ inches and thoroughly washed In screening the material for use in concrete, all fines under No 4 were wasted Coarse aggregate for the concrete was then artificially graded by weight with 60 per cent passing the 1½-inch screen and retained on the ¾-inch screen and 40 per cent passing the ¾-inch and retained on the No 4 screen.

Janesville sand was used as the fine aggregate in all concretes It was well graded to a maximum screen size of No 4 and a fineness modulus of slightly over 3 00. It contained approximately 65 per cent of quartz with dolomite the other principal constituent.

The cement used was a standard brand passing the tests of the A. S. T. M. Standard Specification for Portland Cement (C 9-30) Information on the cement together with certain physical properties of the fine and coarse aggregates are given in Table I.

TESTS ON CONCRETE

Concretes were designed for three water-cement ratios, 0 65, 0 85, and 1 10, by volume, each represented by two slumps, 2 inch and 7 inch, making six kinds of concrete for each of the four coarse aggregates. Design mixes were made in each case, using the sand, crushed limestone and cement in differing proportions to arrive at the mix giving the de-

TABLE I

Physical Properties of Aggregates and Cement

Coarse Aggregates

All coarse aggregate was crushed limestone artificially graded by weight as follows

60% passing 1½-in screen and retained on ¾-in screen
40% passing ¾-in screen and retained on No 4 screen

Sieve Analyses

Material	A	B	C	D
Formation	Trenton	Trenton	Lower Magnesian	Niagara
Sieve size	Percentage of material coarser than sieve			
1-in	3 4	3 4	12 4	7 5
¾-in	32 8	36 6	43 3	41 0
⅜-in	81 3	82 8	83 2	82 5
No 4	97 5	97 4	96 1	97 2
No 8	100 0	100 0	100 0	100 0
Fineness Modulus	7 12	7 17	7 23	7 21

Coarse Aggregate	Weights of Room Dry Rodded Material, in lb per cu ft	Apparent Specific Gravity*	French Coefficient of Wear**	Toughness, Average of 10 tests on 25 by 25 mm cylinders
A Trenton	93 0	2 65	10 1	4 9
B Trenton .	97 3	2 75	9 3	6 4
C Lower Magnesian	97 6	2 72	8 37	7 4
D Niagara	96 9	2 73	12 7	6 6

* A. S T M Standard Method of Test for Apparent Specific Gravity of of Coarse Aggregates, D30–18

** A S T M Standard Method of Test for Test for Abrasion of Rock, D 2–33

Fine Aggregate

Janesville Sand used in all concrete
Weight of room dry, rodded sand—112 9 lb per cu ft
Specific Gravity 2 70
Percentage of Voids 33

Sieve Analysis

Sieve No	Percentage Coarser than Sieve
No 4	2 58
No 8	20 50
No 14	35 66
No 28	55 78
No 48	93 50
No 100	99 31
Fineness Modulus	3 07

TABLE 1—*Concluded*

Cement

Residue on No 200 sieve—95 per cent
Condition of Sample—Satisfactory
Time of set Initial—3 Hr — Min Final—5 Hr — Min
Pat test by steam—Condition of pat satisfactory
Kind of sand used in 1 3 mix—Ottawa Standard
Temperature of water and air—75°F.
Water used for Normal Consistency of cement—23 5%, for 1 3 Mortar—10 4%
Strength of neat cement briquettes at one day—453 lb per sq in

Tensile Strength of 1 3 Mortar Briquettes

Age, days	Lb per sq in
1	182
3	280
7	327
28	408
60	447
180	470
360	388

Chemical Analysis		Compounds Calculated from Chemical Analysis	
CaO	64 00	$CaSO_4$	2 70
MgO	2 39	$4CaO\ Al_2O_3\ Fe_2O_3$	4 65
Al_2O_3	6 65	MgO	2 39
Fe_2O_3	1 53	$3CaO\ Al_2O_3$	15 03
SiO_2	21 74	$3CaO\ SiO_2$	42 96
SO_3	1 59	$2CaO\ SiO_2$	29 99
Loss on Ignition	1 76	Loss on Ignition	1 76
	99 66		99 48
Free CaO	18		

sired water-cement ratio and slump with minimum cement content Economical proportions having been determined for each kind of concrete by this method, the preparation of actual test specimens was started About 500 pounds of concrete was made at one time in a one sack mixer, mixing for 1½ minutes This was sufficient for all specimens of a kind Corrections in the amount of water were necessary at the mixer in many cases to get the desired slump In addition corrections were made for the amount of moisture contained in the aggregates These two items explain the discrepancy between the design and actual values for the water-cement ratios of the various mixes Two types of specimens were made, 4 by 6 by 18-inch beams and 6 by 12-inch cylinders Information on the physical properties of the concrete may be found in Table II Specimens were molded by the writer and an assistant and extreme care was taken to prevent any loss of water After molding they were covered with wet sacks, and the following day, stripped and removed to the moist room All specimens were held in the moist room for at least 28 days

At 28 days all of the compression specimens and one third of the beams were broken The beams were broken on a 16-inch span at a speed of loading head of 0 023 inches per minute

Another third of the beams was removed from the moist room at 56 days, placed in trays and alternately frozen and thawed at the rate

TABLE II

PROPERTIES OF CONCRETE

Rock	Mark	Design Slump in inches	Actual Slump in inches	Proportions by Volume of Room Dry Rodded Material	Water-Cement Ratio by Volume	Cement Content in lb per cu ft of Concrete	Weight of Saturated Concrete from Moist Closet at 28 days, in lb per cu ft
A Trenton	A-1	2	1½	1-1 49-2 70	0 61	26 15	154 3
	A-2	2	2¾	1-2 25-3 59	0 84	19 30	151 9
	A-3	2	2	1-3 08-4 54	1 04	15 35	152 1
	A-4	7	7½	1-1 38-2 04	0 63	29 70	152 0
	A-5	7	6½	1-2 23-2 91	0 84	20 85	149 4
	A-6	7	6½	1-3 12-3 78	1 10	15 90	147 9
B Trenton	B-1	2	2½	1-1 49-2 60	0 63	26 40	156 9
	B-2	2	1¾	1-2 34-3 46	0 79	19 85	155 3
	B-3	2	2¼	1-3 07-4 37	1 02	15 65	155 3
	B-4	7	6½	1-1 38-1 96	0 64	30 55	156 4
	B-5	7	7	1-2 22-2 80	0 85	21 60	154 8
	B-6	7	7	1-3 12-3 65	1 10	16 60	154 7
C Lower Magnesian	C-1	2	1¾	1-1 49-2 58	0 64	26 20	155 6
	C-2	2	2¼	1-2 24-3 44	0 82	19 75	155 0
	C-3	2	1¼	1-3 07-4 35	1 04	15 50	154 0
	C-4	7	7½	1-1 38-1 95	0 64	30 15	154 4
	C-5	7	7¾	1-2 22-2 80	0 84	21 30	152 8
	C-6	7	6	1-3 12-3 63	1 09	16 55	153 7
D Niagara	D-1	2	2¼	1-1 49-2 62	0 64	26 45	157 0
	D-2	2	2	1-2 24-3 48	0 81	19 85	155 8
	D-3	2	2	1-3 06-4 40	1 05	15 60	155 0
	D-4	7	8	1-1 38-1 97	0 63	30 20	154 9
	D-5	7	7½	1-2 22-2 82	0 85	21 60	155 0
	D-6	7	6¼	1-3 12-3 67	1 10	16 55	153 8

of one cycle per day The six inch sides of the beams were vertical and the beams were half immersed in water upon introduction to the freezer and totally immersed while being thawed Freezing temperatures varied from 10°F immediately after placing in the freezer to −8°F 16 hours later Thawing temperatures averaged nearly 60°F These specimens were taken from the freezer, examined and weighed at 25, 50 and 60 cycles At 60 cycles they were broken over a 16-inch span with the side down which had been immersed in water in the freezer

The final third of the beams was removed from the moist room at an age of six months and subjected to a test similar to the above in all respects excepting that the concrete was placed in water only $\frac{1}{4}$ to $\frac{1}{2}$ inch deep when introduced into the freezer These specimens were also broken after 60 cycles of freezing and thawing

A number of specimens were made containing resistance thermometers With these, it was found that the temperature inside of the concrete fell to below freezing within two hours after the specimens, at a temperature of 60°F, were placed in the freezer with its temperature of 10°F The temperature of the concrete then fell more slowly but had reached the freezer temperature of −8°F. at the time of removal, 16 hours later. In thawing, an hour and a half was necessary to bring the interior of the concrete up to within five degrees of the temperature of the thawing water.

TESTS ON ROCK

After washing, a quantity of each of the four crushed rocks was sieved by hand into four sizes This material was allowed to dry in the laboratory for two weeks or longer Separate samples of each size were then made up, about 3000 grams for material larger than $1\frac{1}{2}$-in, 1500 grams for $\frac{3}{4}$ to $1\frac{1}{2}$-in, 750 grams for $\frac{3}{8}$ to $\frac{3}{4}$-in, and 300 grams for No 4 to $\frac{3}{8}$-in These samples were placed in ordinary pint and quart tin cans with holes punched in the lids and the cans were then set in trays for handling into and out of the freezer The samples were thawed under water and the cans were kept full of water on introduction to the freezer so that all rock was continuously immersed. Freezer temperatures, length of cycles, etc, were the same as for the concrete The rock samples were removed from the water at 25, 50 and 100 cycles and allowed to dry in the laboratory for a week, after which each individual sample was sieved by hand through the series of sieves ending with the No. 4. The samples were again immersed and returned to the freezer

Other samples similar to the above were subjected to a modified form of the A S T M Tentative Method of Test for Soundness Coarse Aggregate (C 89–32 T) Sample sizes were as given above The test used differed from the standard in that the material was not oven dried before introduction to the sodium sulfate solution for the first time, and in that, after the completion of washing the sodium sulfate from the rock toward the end of the test, the rock was not then oven dried but was allowed to dry in the laboratory air for at least a week After sieving, this sample was then run through an additional five cycles of the test to make the complete ten cycle test It was found sufficient to dry the rock for four hours between immersions in the sodium sulfate solution. In washing the sodium sulfate from the rock prior to drying and sieving, it was found adequate to wash the rock five times in water as hot as was comfortable to the hand at periods not closer than two hours apart, allowing the rock to remain in the water between washings The addition of a solution of barium chloride to the water from the last washing gave no insoluble precipitate, which demonstrated that the sodium sulfate had been practically removed

Absorption tests were run on both rock and concrete Rock samples consisted of about 1500 grams of room dry material between 1 and $1\frac{1}{2}$ in in size The concrete specimens were halves of the concrete beams which had been broken at an age of 28 days and then stored in the laboratory. Values in Table V for the rock represent the average of at least three tests and for the concrete, two tests

RESULTS OF TESTS

Table III shows the results of strength tests on the concrete cylinders and beams at an age of 28 days Also the strengths are given of the two sets of beams subjected to 60 cycles of freezing and thawing, one set being tested at an age of 22 weeks and the other at 40 weeks These two groups of beams were placed in the freezer for the first time at ages of 56 days and 6 months, respectively A measure of the loss of strength of the concrete due to freezing and thawing is given by the ratio found by dividing the strength after freezing and thawing by the strength at the start of the test To do this it was necessary to estimate the strengths at 56 days and at 6 months on the basis of the 28-days strengths Columns 9 and 10 of Table III contain the results of these calculations, the strength at 56 days being assumed as $1\frac{1}{8}$ of that at 28 days and the strength at 6 months $1\frac{1}{4}$ of that at 28 days These estimates of strength were made after a study of available data as to strength-time relationships for moduli of rupture of concrete beams and it is felt that they are reasonably fair, with the possible exception that the values may be high at six months for the types, A-1, D-1 and D-4 There is little information available showing the strength increase with age of concrete with a modulus of rupture of over 1000 pounds per square inch and it is possible that the assumed strength values at six months are too high in these cases Columns 11 and 12 of Table III show the percentage of strength of the concrete beems retained after 60 cycles of freezing and thawing It should be remembered that the 56-day specimens were half immersed in water while in the freezer and the six-month specimens immersed to a depth of not over $\frac{1}{2}$ inch in water

A number of diagrams are devoted to interpreting the data included in Table III Figure 1 shows the relationship between the strength and water-cement ratio for the concrete cylinders broken at 28 days.

It wıll be noted that although the "A" Trentons show rather lower strengths than the rest of the concretes, all show the ıncrease of strength wıth decrease of water-cement ratıo Furthermore, there ıs no type whıch ıs erratıc ın ıts behavıour Fıgure 2 shows the relatıonshıp

TABLE III

EFFECT OF WEATHERING TESTS ON STRENGTH OF CONCRETE

Rock	Mark	Actual Slump ın inches	Water-Cement Ratıo by Volume	Compressıve Strength of 6 by 12-ın Cylinders ın lb per sq ın at 28 days	Modulus of Rupture of 4 by 6 by 18-ın Beams, ın lb per sq ın: Specımens from Moıst Closet at 28 days	Modulus of Rupture: After 60 cycles of Freezıng and Thawıng Age at Start of Test: 56 days*	Modulus of Rupture: After 60 cycles of Freezıng and Thawıng Age at Start of Test: 6 Mo **	Estımated Modulus of Rupture ın lb per sq ın: At 56 days equal to 1 125 tımes that at 28 days	Estımated Modulus of Rupture ın lb per sq ın: At 6 Months equal to 1 25 tımes that at 28 days	Percentage of Strength Retaıned after 60 cycles of Freezıng and Thawıng Age at Start of test: 56 days Spec Col 7 / Col 9	Percentage of Strength Retaıned after 60 cycles of Freezıng and Thawıng Age at Start of test: 6 Months Spec Col 8 / Col 10
1	2	3	4	5	6	7	8	9	10	11	12
A Trenton	A-1	1½	0 61	6,000	1,060	855	1,090	1,190	1,325	72	82
	A-2	2¾	0 84	3,890	840	535	735	945	1,050	57	70
	A-3	2	1 04	2,410	660	40	125	745	825	5	15
	A-4	7½	0 63	5,650	885	765	820	995	1,105	77	74
	A-5	6⅓	0 84	3,440	735	95	245	825	920	11	27
	A-6	6½	1 10	1,880	575	0	15	645	720	0	2
B Trenton	B-1	2½	0 63	6,600	1,000	875	990	1,125	1,250	78	79
	B-2	1¾	0 79	4,250	860	490	590	965	1,075	51	55
	B-3	2¼	1 02	2,540	640	40	125	720	800	6	16
	B-4	6½	0 64	6,550	1,010	830	1,035	1,135	1,265	73	82
	B-5	7	0 85	4,320	845	220	400	950	1,055	23	38
	B-6	7	1 10	2,220	635	60	90	715	795	8	11
C Lower Magnesıan	C-1	1¾	0 64	6,370	1,015	980	1,055	1,140	1,270	86	83
	C-2	2¼	0 82	4,470	880	295	615	990	1,100	30	56
	C-3	1¼	1 04	2,370	775	325	400	870	970	37	41
	C-4	7½	0 64	6,900	990	975	1,095	1,115	1,240	87	88
	C-5	7¾	0 84	4,520	855	405	775	960	1,070	42	72
	C-6	6	1 09	2,320	670	80	160	755	835	11	19
D Nıagara	D-1	2¼	0 64	6,920	1,045	1,150	1,195	1,175	1,310	98	91
	D-2	2	0 81	4,470	935	730	885	1,050	1,170	70	76
	D-3	2	1 05	2,310	725	385	525	815	905	47	58
	D-4	8	0 63	6 770	1,055	1,155	1,165	1,185	1,320	98	88
	D-5	7½	0 85	3,810	880	450	785	990	1,100	45	71
	D-6	6¼	1 10	2,400	625	160	420	705	780	23	54

* Broken at age of 22 weeks In moıst closet 7 to 9 days after completıon of weatherıng test

** Broken at age of 40 weeks In moıst closet 7 to 9 days after completıon of weatherıng test

Each value represents the average of three tests between the moduli of rupture and water-cement ratio for the concrete beams broken at an age of 28 days Here the concretes of 7-inch slump using "A" Trenton as a coarse aggregate are quite low in strength again For medium and low water-cement ratios the strengths of the remainder of the concretes are reasonably close but in the case of the high water-cement ratios there is considerable scattering of strengths

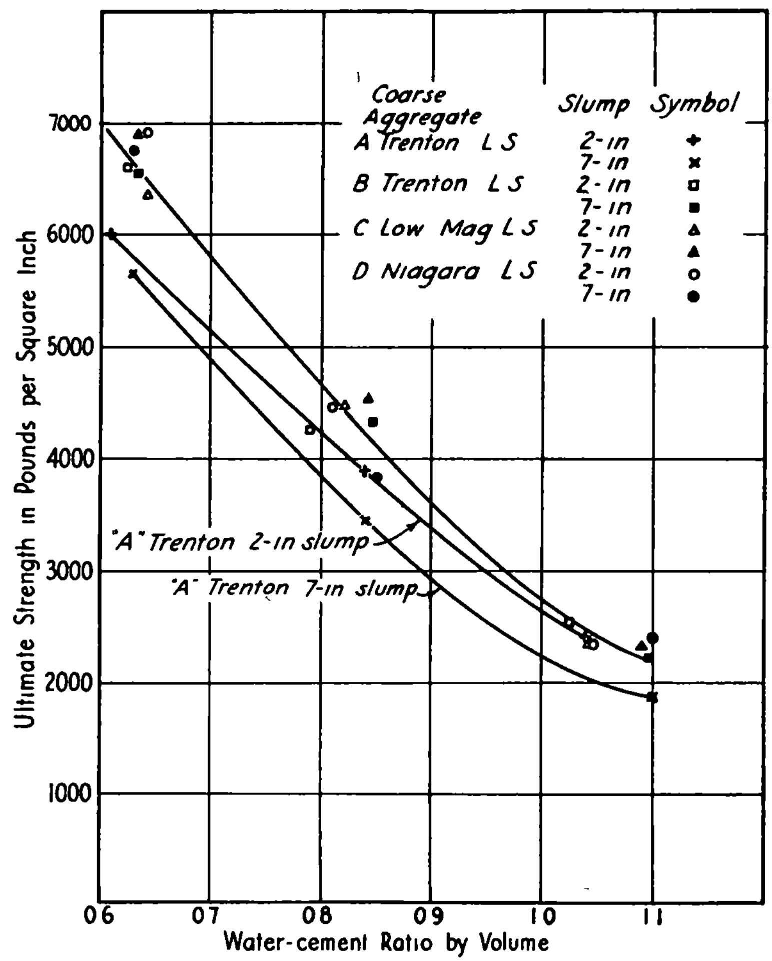

Figure 1 Compression Strength of Concrete Cylinders at 28 Days

The "C" Lower Magnesian of 2-inch slump (C-3) shows an unusually high strength as compared with the others whereas the "B" Trenton with 2-inch slump is rather low The writer can assign no definite reason for this, especially as the compressive strengths of cylinders made from the same batches were perfectly normal

Figure 3 shows the effect of amount of mixing water and kind of coarse aggregate on the resistance of the concretes to freezing and thawing. Each point on this diagram represents 12 specimens which include the 56-day and the 6-month specimens of both 2 and 7-inch slumps The decrease of resistance to freezing and thawing with an increase of water-cement ratio is very clearly and definitely shown There is also a clearly shown banding of the concretes according to the type of coarse aggregate used Data subsequently presented show that durability tests on the coarse aggregates graded them, in order of increasing resistance to weathering, A, B, C, and D, with small

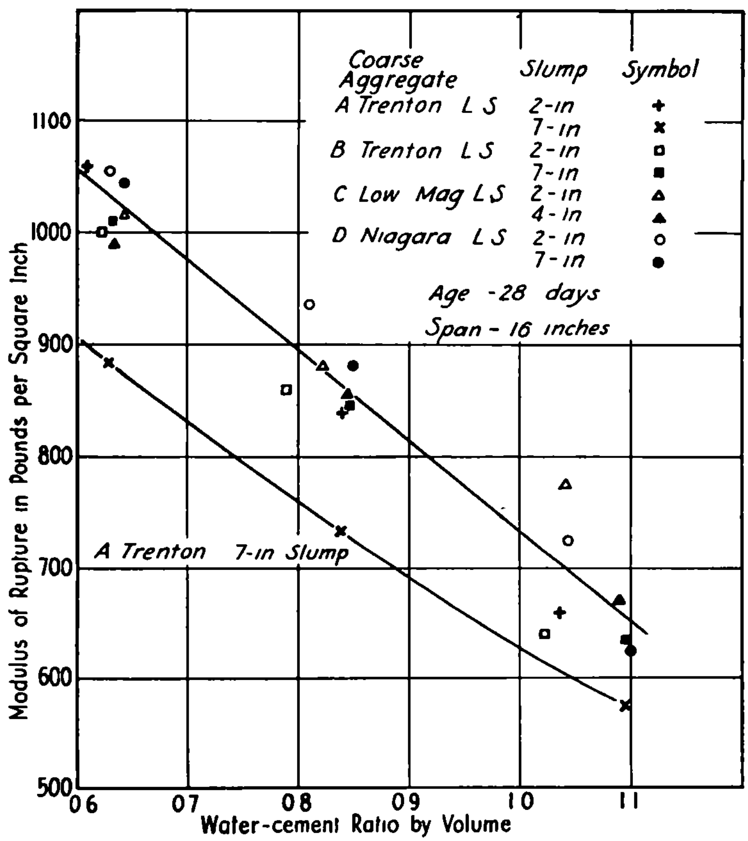

Figure 2. Cross Bending Tests on 4 x 6 x 18-in. Concrete Beams

differences between A and B The freezing and thawing tests on the concrete show the same order so that there is a real basis for believing that, as far as crushed limestone aggregates are concerned, the durability of the aggregate is reflected in the durability of the concrete It must be remembered that the rock was all clean and fresh and washed before use, and while A and B would not rate as high under visual inspection as C and D, they were far from being "rotten"

Figure 4 shows the effects of age and type of test on the resistance of the concretes to weathering Each point represents six specimens, which include those of 2 and 7-inch slump In one way it is unfortunate that it was found necessary to reduce the depth of immersion for the six-month specimens to one-half inch while in the freezer in order to cut down the freezer load The 56-day specimens were immersed in three inches of water, and this makes it impossible to tell just how much of the spread between the solid and dotted lines representing 6-month and 56-day specimens is due to age and how much is

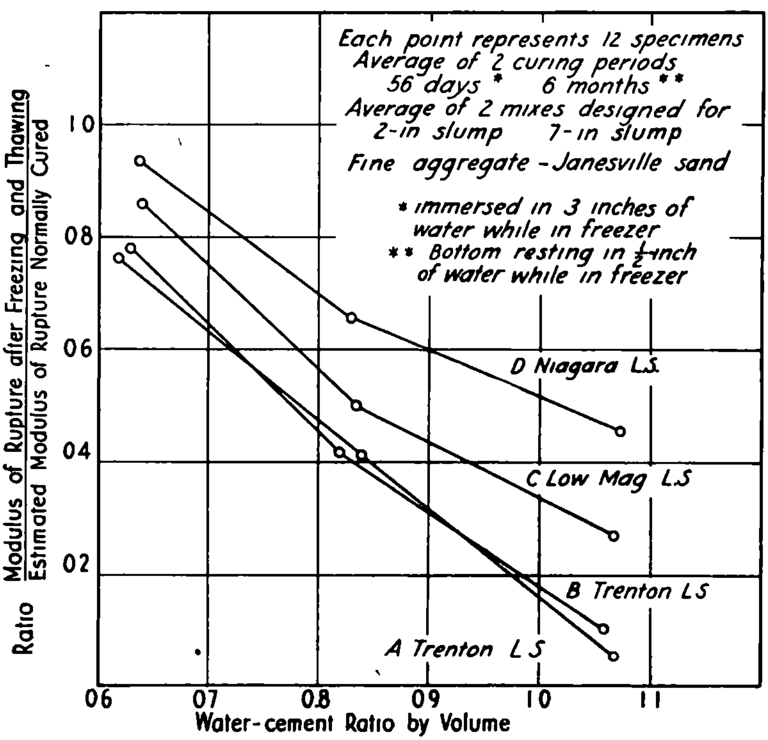

Figure 3 Effect of 60 Cycles of Freezing and Thawing on 4 x 6 x 18-in Concrete Beams

due to type of test The writer is of the opinion that, inasmuch as the bottoms of the specimens were immersed in each case and there was water available in the tray until it froze, that age was the major factor in making the six-month specimens the more resistant to the weathering test The curves are by no means as regular as those in Figure 3 and it is thought that a reason for this may be the smaller number of specimens represented per point. The data for the concrete of "D" Niagara rock show that the 56-day concrete of low water-cement ratio was more resistant than the 6-month concrete of the same water-cement ratio It is thought that these ratios in Figure 4 may not truly represent these ages, on account of lack of a good basis for estimating the increase in the modulus of rupture of such high strength concrete due to ageing from 28 to 56 days or to 6 months

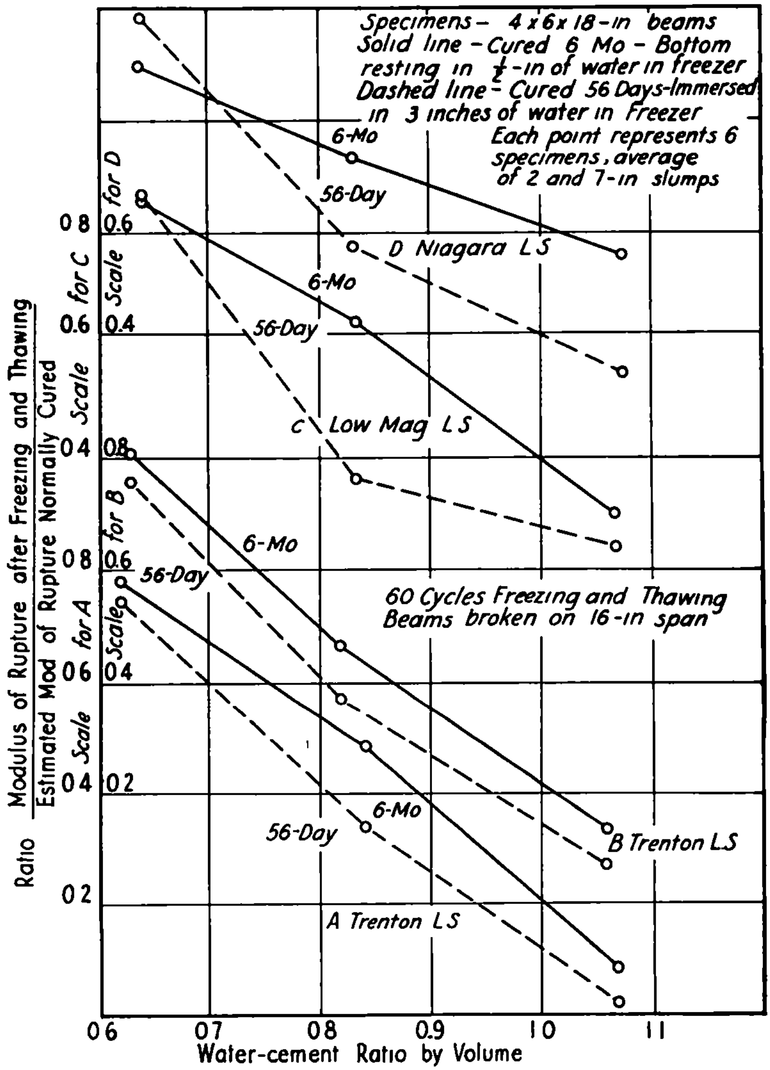

Figure 4. Effect of Age and Type of Test on Resistance of Concrete to Freezing and Thawing

Figure 5 shows the effect of consistency on the resistance of concrete to weathering The general appearance of the curves suggests that low slump concretes are somewhat more resistant than high slump concretes Each point represents six specimens made at the same time A thorough study of the original data from these tests reveals no conclusive reason for the erratic behaviour of a few of the concretes represented on this curve sheet

Of the four variables above—age, consistency, amount of mixing water, and type of coarse aggregate—the latter has been studied at

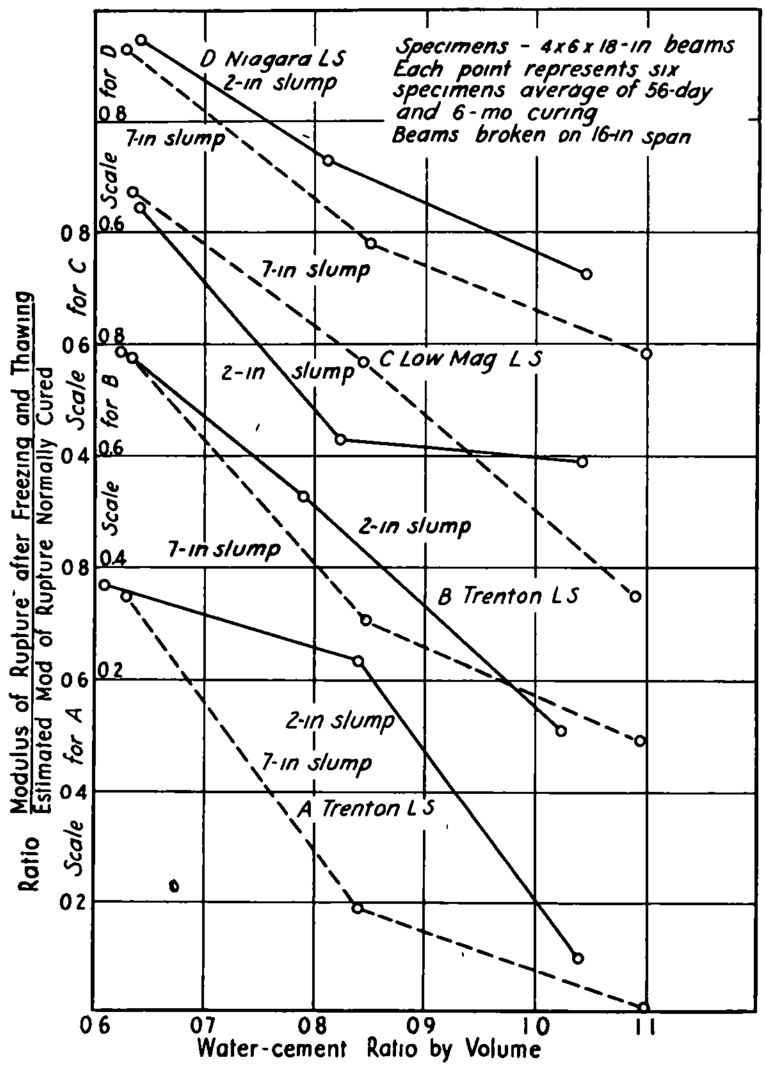

Figure 5. Effect of Consistency on Resistance of Concrete to 60 Cycles of Freezing and Thawing.

considerable length Table IV shows the effect of 25, 50 and 100 cycles of freezing and thawing and 5 and 10 cycle runs of a modified form of the A S T. M Soundness Test in reducing the size of the crushed rock particles This effect is measured in two ways The first or "fineness index" method amounts to finding a modified fineness modulus for the sample before and after the weathering or soundness test This modified fineness modulus is found in the ordinary way with the exception that only four sieves, the $1\frac{1}{2}$-in, the $\frac{3}{4}$-in, the $\frac{3}{8}$-in and the No 4 are used in actual sieving and considered in computing the results

TABLE IV

EFFECT OF WEATHERING TESTS IN REDUCING SIZE OF CRUSHED ROCK PARTICLES

Rock	Size	Fineness* Index Before Test	Fineness Index* After Test					Percentage Loss** After Test				
			Freezing and Thawing			A S T M Sodium Sulfate Test (Modified)		Freezing and Thawing			A S T M Sodium Sulfate Test (Modified)	
			25 Cycles	50 Cycles	100 Cycles	5 Cycles	10 Cycles	25 Cycles	50 Cycles	100 Cycles	5 Cycles	10 Cycles
A Trenton	(a) No 4–$\frac{3}{8}$	1 00	0 93	0 84	0 78	0 94	0 63	7 3	15 6	21 6	6 2	37 6
	(b) $\frac{3}{8}$–$\frac{3}{4}$	2 00	1 88	1 76	1 61	1 92	1 53	8 5	16 9	26 6	5 4	30 7
	(c) $\frac{3}{4}$–$1\frac{1}{2}$	3 00	2 80	2 65	2 44	2 78	2 22	12 9	21 7	37 3	13 6	37 8
	(d) $1\frac{1}{2}$+	4 00	3 74	3 38	3 01	3 42	2 84	16 8	44 2	60 9	32 7	57 4
*** Entire Sample		2 70	2 53	2 34	2 13	2 45	1 96	12 0	25 2	38 2	15 2	40 9
B Trenton	No 4–$\frac{3}{8}$	1 00	0 94	0 89	0 85	0 96	0 76	6 4	10 9	14 7	4 3	25 9
	$\frac{3}{8}$–$\frac{3}{4}$	2 00	1 81	1 69	1 60	1 95	1 61	13 3	21 9	27 3	4 5	25 9
	$\frac{3}{4}$–$1\frac{1}{2}$	3 00	2 83	2 72	2 56	2 74	2 46	8 8	17 7	26 4	16 1	28 0
	$1\frac{1}{2}$+	4 00	3 78	3 50	3 26	3 50	2 99	12 6	28 5	34 6	28 8	45 4
Entire Sample		2 70	2 53	2 38	2 24	2 46	2 13	10 5	20 4	26 9	14 6	31 2
C Lower Magnesian	No 4–$\frac{3}{8}$	1 00	0 97	0 93	0 88	0 98	0 91	3 4	6 7	11 7	2 3	9 3
	$\frac{3}{8}$–$\frac{3}{4}$	2 00	1 94	1 88	1 82	1 98	1 90	3 6	8 1	11 7	1 0	6 1
	$\frac{3}{4}$–$1\frac{1}{2}$	3 00	2 94	2 89	2 80	2 98	2 92	3 2	6 7	11 4	1 5	4 3
	$1\frac{1}{2}$+	4 00	3 94	3 89	3 78	3 93	3 70	4 3	7 2	15 7	4 1	18 2
Entire Sample		2 70	2 65	2 59	2 51	2 66	2 56	3 6	7 2	12 6	2 2	9 0
D Niagara	No 4–$\frac{3}{8}$	1 00	1 00	0 99	0 98	0 98	0 96	0 4	0 8	1 4	1 6	3 8
	$\frac{3}{8}$–$\frac{3}{4}$	2 00	1 98	1 97	1 96	1 98	1 97	1 3	3 0	3 7	1 5	3 1
	$\frac{3}{4}$–$1\frac{1}{2}$	3 00	2 99	2 99	2 97	2 98	2 98	0 4	0 7	1 8	1 3	1 6
	$1\frac{1}{2}$+	4 00	3 99	3 98	3 93	3 99	3 95	0 7	1 6	5 0	0 6	3 6
Entire Sample		2 70	2 69	2 69	2 66	2 69	2 67	0 7	1 5	3 0	1 2	2 8

* The "Fineness Index" is determined by adding percentages of the rock retained on No 4, $\frac{3}{8}$-in, $\frac{3}{4}$-in, and $1\frac{1}{2}$-in sieves

** "Percentage Loss" is the amount in per cent of the rock passing, after the test, the sieve on which it was originally retained

*** Values for "Entire Sample" found by weighting the four sizes, a, b, c, d, 15, 25, 35, and 25 per cent respectively

Each value represents the average of 3 tests

The second method of measuring the effect is by the "percentage loss" or that percentage of material passing, after the test, the sieve on which it was all retained before the test In computing the average values for the entire sample, the different sizes were weighted according to their use in a typical crushed rock concrete for road pavement

Figure 6 correlates the results of the various accelerated weathering tests on the rock samples with results of freezing and thawing tests on concrete beams The strength ratios for the beams are plotted as

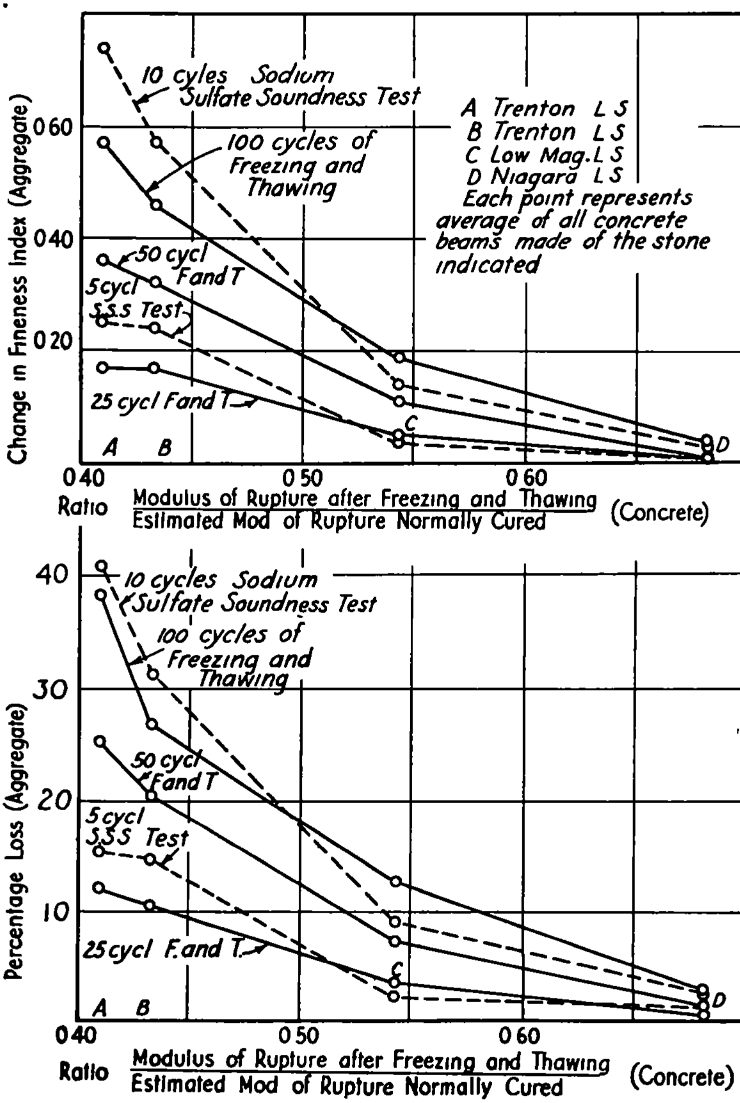

Figure 6. Effect of Various Weathering Tests on Four Crushed Limestone Aggregates

abscissas with the fineness indices or percentages of loss for the corresponding rocks as ordinates. By comparing the ordinate values for each accelerated test it can be seen that all of the tests classify the rock in the same manner. By comparing dash and full lines it will be observed that 5 cycles of the A S T M Soundness Test are approximately comparable to 25 cycles of freezing and thawing in severity and 10 cycles of the former are nearly equivalent to 100 cycles of the latter It is interesting to note the pronounced disintegration of the poorer rocks caused by the A S T M Soundness Test The slopes of all curves in Figure 6 plainly indicate that lack of durability of a limestone rock, as measured by these weathering tests, adversely affects the resistance to freezing and thawing of the concrete containing the rock

Figure 7 is a comparison of the two ways of measuring the disintegrating effects of the accelerated weathering tests on a rock The straight line relationship shows that these two methods are in accord for the crushed limestones tested This suggests the sole use of per-

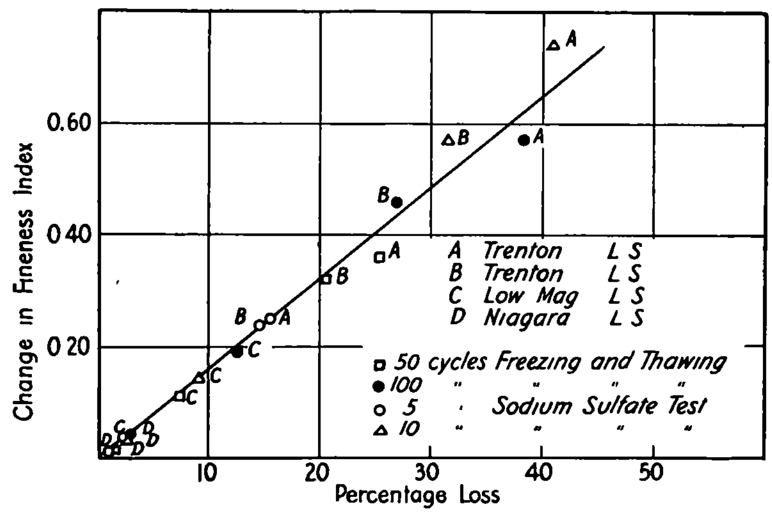

Figure 7. Comparison of Methods of Measuring Effects of Weathering Tests on Coarse Aggregates

centage loss for a measure of the effect of such tests, inasmuch as it involves less labor in sieving then does the determination of the necessary data for calculating the fineness index of the material Figure 8 shows the percentage loss of each of the different sizes for the four kinds of rock after 100 cycles of freezing and thawing While there is a tendency towards spread, it can be seen that with one single exception the percentage loss for any of the sizes tends to classify the rock as to weathering resistance, and this one exception only fails to distinguish between the two poorest rocks, A and B The percentage loss for the material over 1½ inch in size shows exactly the same trend as the percentage loss for the entire sample but exaggerates it for the poorer rock The whole of the evidence given here illustrates the possibility of using percentage loss on material of a fairly large size as a criterion of the relative degree of weathering resistance of the various rocks Such a measure would reduce the amount of labor involved in making accelerated weathering tests and lessen the possibility of errors, as it is a rapid matter to sieve material of a large size and there is small likelihood of losing it at any stage of the test This point is now under investigation for about forty limestone aggregates at the University of Wisconsin

Table V shows the results of absorption tests on room dry concrete and crushed rock This material was not oven dried at any time For any given coarse aggregate, the concrete shows an increase in amount of absorption as the water-cement ratio increases Between the various concretes, however, there is no consistent relationship to be observed between absorption or rate of absorption and resistance to freezing and thawing No further studies have been included bear-

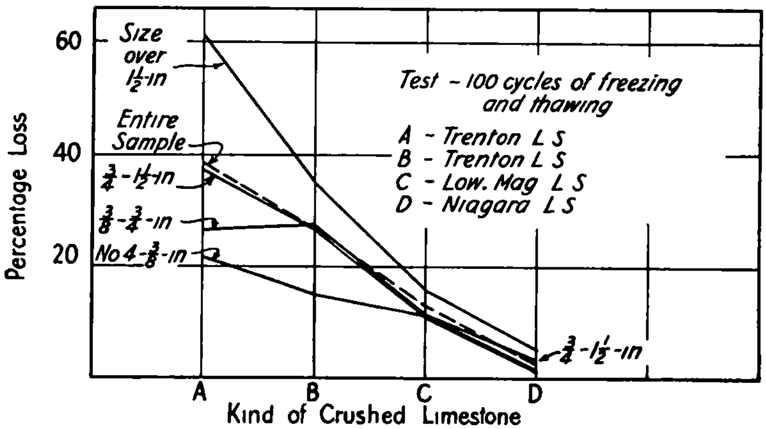

Figure 8 Effect of Freezing and Thawing on Different Sized Particles of Coarse Aggregates

ing on this subject as a short examination of Tables III and V will show that, in many cases, concretes of about the same absorption and rate of absorption had widely differing resistances to freezing and thawing Considering the absorption data for the crushed rock, it is noteworthy that the A and B Trentons show a smaller rate of absorption as determined by the ratio of 6 minute absorption to 24 hour absorption than do the more resistant rocks C and D

Table VI shows the percentage loss in weight of the concrete beams at various stages of the freezing and thawing tests In addition, the last column of the table gives the condition of the 56-day specimens as determined by visual inspection after 60 cycles of freezing and thawing Each set of specimens was placed in one of six classes according to the amount of disintegration observable In doing this, the writer assigned each type to its particular group after reference to pictures and written descriptions of the specimens and disregarded any information available as to strength losses It will be noticed that the correlation between their condition and loss of weight was very good, which is only natural, as any considerable loss in weight leaves a sorry looking speci-

TABLE V

ABSORPTION TESTS ON CONCRETE AND CRUSHED ROCK

Tests on Concrete

Rock	Mark	Actual Slump in inches	Water-Cement Ratio by Volume	Percentage Absorption by Weight for Room Dry* Concrete at age of 23 Weeks						Ratios	
				6 Min	15 Min	1 Hr	6 Hrs	24 Hrs	48 Hrs	6 Min / 24 Hr	1 Hr / 24 Hr
A Trenton	A-1	1½	61	23	31	52	91	1 28	1 44	18	41
	A-2	2¾	84	54	78	1 20	1 95	2 49	2 76	22	48
	A-3	2	1 04	71	95	1 45	2 35	2 95	3 23	24	49
	A-4	7½	63	27	42	71	1 24	1 66	1 88	16	43
	A-5	6½	84	41	62	1 06	1 85	2 47	2 79	17	43
	A-6	6½	1 10	73	1 07	1 79	2 93	3 78	4 07	19	47
B Trenton	B-1	2½	63	30	42	71	1 20	1 56	1 74	19	46
	B-2	1¾	79	38	57	99	1 63	2 16	2 42	18	46
	B-3	2¼	1 02	68	96	1 58	2 66	3 43	3 67	20	46
	B-4	6½	64	36	53	87	1 45	1 88	2 10	19	46
	B-5	7	85	44	62	1 03	1 73	2 20	2 44	20	47
	B-6	7	1 10	64	89	1 46	2 41	3 03	3 26	21	48
C Lower Magnesian	C-1	1¾	64	38	51	77	1 19	1 44	1 68	26	53
	C-2	2¼	82	56	77	1 20	1 89	2 39	2 58	23	50
	C-3	1¼	1 04	76	1 06	1 66	2 55	3 10	3 28	25	54
	C-4	7½	64	41	58	93	1 40	1 72	1 89	24	54
	C-5	7¾	84	57	80	1 23	1 86	2 34	2 52	24	53
	C-6	6	1 09	82	1 15	1 85	3 06	3.76	3 94	22	49
D Niagara	D-1	2¼	64	28	42	67	98	1 25	1 37	22	54
	D-2	2	81	41	54	90	1 43	1 83	2 00	22	49
	D-3	2	1 05	68	95	1 52	2 33	2 82	2 97	24	54
	D-4	8	63	35	49	81	1 22	1 52	1 65	23	53
	D-5	7½	85	54	77	1 18	1 84	2 28	2 44	24	52
	D-6	6¼	1 10	61	92	1 54	2 63	3 31	3 52	18	47

Tests on Rock**

Rock	6 Min	15 Min	1 Hr	6 Hrs	24 Hrs	48 Hrs	6 Min / 24 Hr	1 Hr / 24 Hr
A Trenton	39	58	88	1 23	1 30	1 33	30	68
B Trenton	28	43	57	75	77	82	36	74
C Low Mag	56	61	71	83	89	95	63	80
D Niagara	29	34	42	50	55	60	53	76

* Stored for 19 weeks in Laboratory—Ave Temp 75°F, Ave Humidity Approx 40%

** Rock stored in laboratory for more than one year Percentage absorption based on weight of room dry material Temperature and humidity approximately as above.

men The correlation between condition and loss of strength, while good for the specimens in the best and poorest groups, is only fair for the intermediate groups Weight losses tend to be either extreme or

TABLE VI

LOSS OF WEIGHT OF CONCRETE IN WEATHERING TESTS

Rock	Mark	Actual Slump in Inches	Water-Cement Ratio by Volume	Percentage Loss of Weight During Freezing and Thawing Tests — 56 day Specimens at end of 25 Cycles	50 Cycles	60 Cycles	6 Mo Spec at End of 60 Cycles	Condition* of 56 day Specimens at 60 cycles as determined by Visual Inspection
A Trenton	A-1	1½	61	0 1	0 1	0 2	2	1
	A-2	2¾	84	0 2	1 0	1 7	2	3
	A-3	2	1 04	2 8	25 6	57 5	6 4	6
	A-4	7½	63	0 1	0 3	0 3	3	1
	A-5	6½	84	0	1 4	2 8	2	3
	A-6	6½	1 10	2 3	62 6	100 0	50 2	6
B Trenton	B-1	2½	63	0 1	0 1	0 2	2	1
	B-2	1¾	79	0	0 4	0 5	6	2
	B-3	2¼	1 02	0 3	12 6	19 1	2 1	5
	B-4	6½	64	0 1	0 1	0 1	1	1
	B-5	7	85	0	0 5	1 0	4	3
	B-6	7	1 10	0 3	7 3	15 9	5 9	5
C Lower Magnesian	C-1	1¾	64	0	0 1	0 1	2	1
	C-2	2¼	82	0	1 8	3 1	1	4
	C-3	1¼	1 04	0 6	3 8	6 2	2 3	4
	C-4	7½	64	0 1	0 1	0 1	3	1
	C-5	7¾	84	0	0 5	0 8	5	2
	C-6	6	1 09	0 7	12 4	20 3	4 7	5
D Niagara	D-1	2¼	64	0 1	0 1	0 1	2	1
	D-2	2	81	0	0 3	0 7	4	2
	D-3	2	1 05	0	1 8	2 9	6	3
	D-4	8	63	0 1	0 1	0 1	2	1
	D-5	7½	85	0	0	0 2	3	2
	D-6	6¼	1 10	0	2 6	5 7	1 1	4

* Notes on condition of 4 by 6 by 18-in beam specimens

1 No appreciable disintegration

2 Slight disintegration of mortar on top and side surfaces with slight beveling of top corners and edges

3 Some disintegration of mortar on top and side surfaces with considerable spalling of concrete on upper corners

4 Mortar spalling rather generally on top and side surfaces Checking visible on all surfaces Top corners disintegrated with this condition likely extending over ends

5 Top, sides and ends all spalling badly with considerable loss of material but with a portion of the specimen still solid

6 Nearly or completely disintegrated Can be largely broken up with bare hands

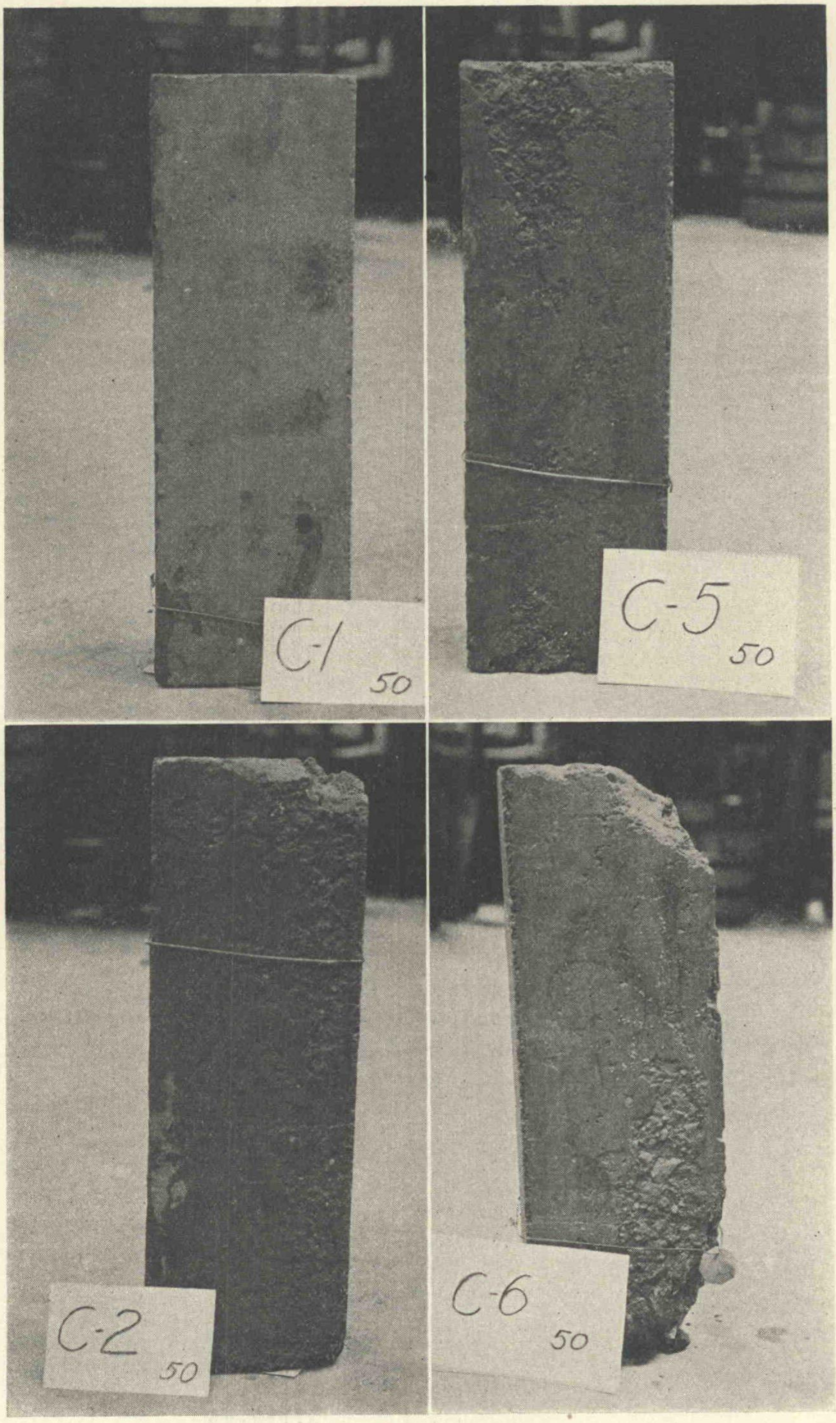

Fig. 9

negligible Because of this, the percentage loss of strength has been made the basis of classification as to relative durability of the concretes studied in this paper As an illustration of the degree of severity of the freezing and thawing test, and also as a visual explanation of the grading given at the bottom of Table VI, pictures of four specimens of concrete made using "C" Lower Magnesian rock are shown in Figure 9

EXPANSION TESTS ON CONCRETE BEAMS

Incidental to the work described above, tests were run on some twenty concrete beams to determine their expansion due to alternate freezing and thawing These beams had been made earlier by Messrs J H Craven and P R Ferguson in connection with another research project, but, as room was available in the freezer, it was thought that some expansion tests might well be included Full information on the physical properties of the beams is given in Table VII Freezing conditions were similar to those of the specimens tested for strength, the only difference being that the expansion beams were not immersed while in the freezer Approximately every five cycles, during the thawing period, each of the specimens was measured with a 20-inch strain gage to get the change in length It will be noted that the principal variables are water-cement ratio, slump, proportions of mix, curing conditions and kind of cement used

With the above number of variables and 20 specimens, it is of course impossible to make a complete study of the effect of each on the expansion The effects of certain of the variables can be seen in Figure 10, however, in which unit expansion is plotted against cycles of freezing and thawing Three things are evident from a study of this figure Those beams made using B brand of quick hardening cement showed a higher rate of expansion than any other of the specimens This may be partially explained by the fact that the concrete in these beams had a rather low slump and may have inclined to harshness with resulting low density However, the concrete in beams using "A" brand of quick hardening cement in some cases had low slumps but did not show

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Figure 9 Effect of 50 cycles of freezing and thawing on four concrete specimens made using "C" Lower Magnesian limestone as a coarse aggregate. These specimens were cured 56 days before they were subjected to the weathering test On the basis of the grading used in Table VI, these specimens would be rated in increasing order of disintegration as follows: C-1 in class 1, C-5 in class 2, C-2 in class 3 and C-6 in class 5.

Mark	Water-Cement Ratio by Vol	Slump in inches
C-1	0 64	$1\frac{3}{4}$
C-2	0 82	$2\frac{1}{4}$
C-5	0 84	$7\frac{3}{4}$
C-6	1 09	6

this high expansion In many ways the beams using "A" brand behaved much like those using portland cement A second point to be

TABLE VII

EXPANSION TESTS ON CONCRETE BEAMS SUBJECTED TO FREEZING AND THAWING

Data on Mixes

No	Water-Cement Ratio by Vol	Slump in inches	Proportions of Mix by Weight	Curing Conditions			Cement Content in lb per cu ft of concrete
				Moist Closet, days	Air, days	Moist Closet, days	
1A	0 99	1	1 3 95 3 95	7	70	292	16 90
7A	0 98	1	1 3 95 3 95	7	70	278	17 20
1P	0 80	1	1 2 68 2 89	7	70	292	22 90
7P	0 80	3¼	1 2 68 2 89	7	70	278	23 20
1B	0 92	¾	1 3 95 3 95	7	70	292	16 60
7B	0 93	0+	1 3 95 3 95	7	70	278	16 10
2A	0 65	1½	1 2 25 2 78	7	70	291	25 35
8A	0 65	1	1 2 25 2 78	7	70	271	25 45
2P	0 51	3	1 0 85 1 40	7	70	291	46 50
8P	0 51	2	1 0 89 1 40	7	70	271	47 00
2B	0 57	¾	1 1 86 2 54	7	70	291	28 70
8B	0 57	¾	1 1 86 2 54	7	70	271	28 15
3A	0 89	1¼	1 3 51 3 51	7	70	290	18 75
3P	0 68	1¼	1 2 43 2 86	7	70	290	24 50
10A	0 80	½	1 3 21 3 60	7	70	265	19 60
10P	0 57	2	1 1 75 2 34	7	70	265	30 75
11A	1 01	6	1 2 91 3 26	328			22 00
11P	0 80	3	1 2 68 2 89	348			23 55
12A	0 65	6½	1 1 82 2 14	328			31 60
12P	0 50	7	1 1 00 1 25	341			48 00

A—Quick hardening cement
B—Quick hardening cement
C—Portland cement
4 by 4½ by 21-in concrete specimens measured over 20-in gage length
Janesville sand and gravel used as aggregate
Concrete not immersed while in freezer

Data on Temperatures

Concrete temp. on introduction to freezer 60°F (Approx).
Freezer temp on introduction of concrete 10°F (Approx)
Both fall to −8°F in 16 hours or less

Concrete thawed in water at 60°F (approx) for 8 hours
Concrete reaches temp of 55°F (approx) in 1½ hours

observed is that the four specimens of high slump cured continuously in the moist room showed much higher rates of expansion than did the other beams of "A" and portland cements, which were all of lower slumps and which had had some air curing Both the method of curing and the consistency apparently had some effect here Lastly, given two beams of about the same consistency and using the same brand

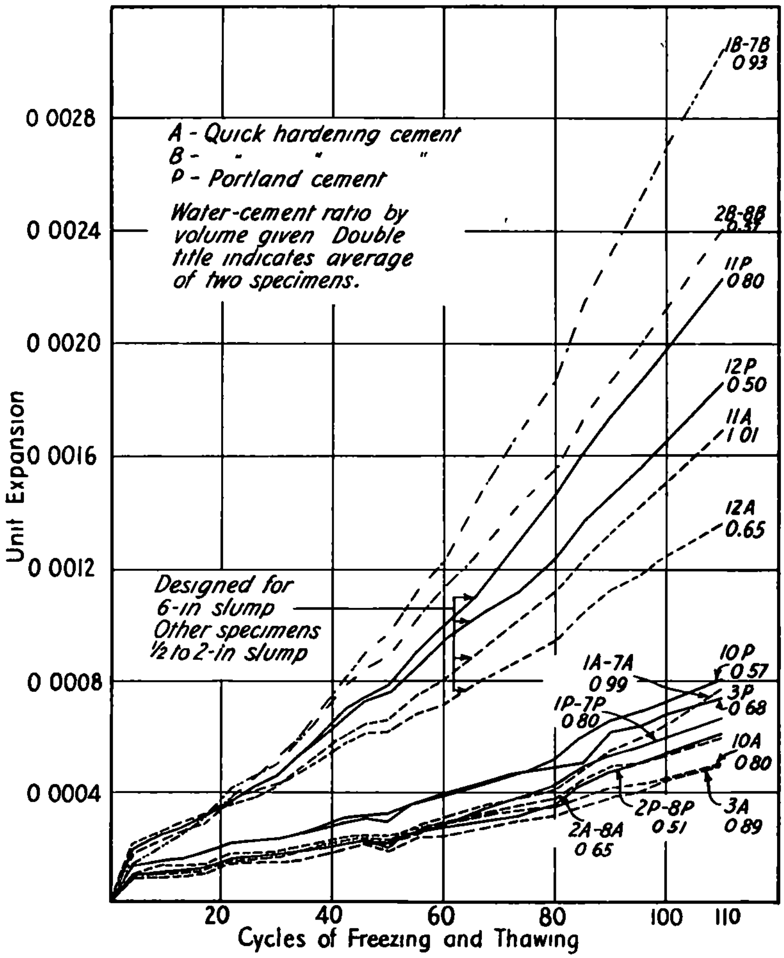

Figure 10 Expansion of 4 x 4½ x 21-in Concrete Beams Due to Freezing and Thawing

of cement, the one with the higher water-cement ratio generally showed a slightly greater rate of expansion

The eight beams which had a unit expansion of over 0 001 were all showing some visual evidence of disintegration due to freezing and thawing Beams 1B and 7B (unit expansion, 0 003) were badly disintegrated

SUMMARY

1 For the concretes using the four crushed limestones of differing characteristics as the coarse aggregates

a The resistance of the concrete to freezing and thawing was found to decrease directly as the water-cement ratio increased

b The quality of the coarse aggregate as determined by any of the given accelerated weathering tests was directly reflected in the resistance to freezing and thawing of the concrete

c The resistance of the concrete to freezing and thawing was somewhat increased by an additional length of curing prior to the weathering test.

d. Sixty cycles of freezing and thawing with partial immersion of the type used proved a severe weathering test for concrete cured in a moist room

e. Strength losses due to freezing and thawing were not always accurately indicated by loss in weight or condition as determined by visual inspection, although the last two measures agreed very well between themselves

f No definite relationship was found to exist between the absorptive properties and the resistance to freezing and thawing of the concretes tested

2 For four crushed limestone coarse aggregates:

a Freezing and thawing tests and a slightly modified form of the A. S T M Tentative Method of Test for Soundness of Coarse Aggregate both graded the rocks in the same relative order of durability or resistance to weathering

b The reduction in particle size due to 25 and 100 cycles of freezing and thawing was roughly duplicated by that due respectively to 5 and 10 cycle runs of a slightly modified form of the A S T M Soundness Test

c. Change in either "fineness index" or "percentage loss" served equally well as measures of the amount of disintegration caused by weathering tests

d. Evidence was developed that a measure of weathering resistance is given by the percentage loss of the large size rock particles alone

DISCUSSION

ON

WEATHERING TESTS ON CONCRETE

Mr A T. Goldbeck, *Director, Bureau of Engineering, National Crushed Stone Association.* In discussing Mr Hanson's excellent paper, I find myself in the very undesirable position of wishing that it were possible for me to heartily disagree with his test results, but finding, upon analyzing them, that I can offer no major criticisms I do not think Mr Hanson would imply that his results are anything more than indicative They cannot be conclusive because of the comparatively small number of test specimens used for each mixture of concrete and, further, because of the very limited number of limestones tested

It is very clearly indicated by these tests that the durability of concrete is controlled largely by the water-cement ratio of the mixture, but there is one point I should like to bring out in this connection which Mr Hanson has not touched on to any great extent

It will be recalled that with each stone he made six mixtures, two of them with a low water-cement ratio, two with a medium and two with a high water-cement ratio The mixtures were so designed that three of them had a rather dry consistency as indicated by the slump test and three of them a comparatively wet consistency The water-cement ratios used in the wet consistency concretes were almost identical with those used in the dry consistency concretes Although Mr. Hanson does not show the cement factor used in the various mixes, it must be clear that in order to obtain high slump in one case and low slump in another, more cement paste must have been used in the wet consistency mixtures than in those of dry consistency, and, therefore, a greater volume of water must have been used in those mixtures having a high slump than in those having a low slump.

As cement requires a comparatively small amount of water for hydration, roughly two gallons per sack there must have been left over a larger amount of free water in the case of the wet mixtures than in the case of the dry mixtures This free water creates porosity, and higher porosity in the wet mixtures than in the dry In all probability this will explain why the wet mixtures disintegrated to a greater extent than the dry mixtures in the freezing tests, notwithstanding the fact that they had the same water-cement ratio as the dry mixtures

These results are quite in line with those obtained in our laboratory a few years ago We found that the free water, which creates porosity, is a somewhat better index of the resistance of the concrete to weathering than is the water-cement ratio

Mr Hanson's results seem to indicate that rocks which show low resistance in accelerated tests, whether freezing and thawing or sodium sulfate tests, seem to bring about more rapid disintegration in concrete than rocks which are more resistant This indication seems to be quite reasonable However, an analysis of Mr Hanson's curves indicates that a rock having comparatively low resistance in the accelerated test may give just as good strength results as a rock of high resistance if it is used with a low water-cement ratio concrete

Mr Hanson speaks of the desirability of using the percentage of loss of weight of the rock sample after a given number of cycles of freezing and thawing as a proper index of the weathering resistance of that rock, the same sieve being used in the determination of percentage of loss as in the preparation of the sample In this connection I should like to call attention to the fact that a serious error may be introduced in the determination of the percentage of loss by this procedure It may well happen that, due to the test, fragments of rock, essentially sound, may become spalled off so that they then will pass the sieve opening used in the preparation of the sample These sound pieces would be accounted as unsound in the procedure suggested by Mr Hanson, where, as a matter of fact, these fragments may be entirely sound

Let me illustrate what I mean by some test results obtained in our laboratory on a sample of limestone The $2\frac{1}{2}$ inch to $1\frac{1}{2}$ inch fraction after the sodium sulfate test, was sieved through the $1\frac{1}{2}$ inch screen It had a loss of 40 per cent, when sieved through the 1 inch screen, it had a loss of only 9 per cent, and when sieved through the $\frac{3}{4}$ inch screen, a loss of only $8\frac{1}{2}$ per cent The other fractions gave similar results The large fragments which passed through the $1\frac{1}{2}$ inch screen were not unsound any more than the fragments which were retained on this screen They were merely spalled off so that they now could pass through this size of opening

The American Society for Testing Materials' specifications are defective in this particular method of determining the percentage of loss after the test and it is my suggestion that it could be made into a satisfactory specification through the use of a screen having an opening one-half the size of the opening of the minimum size screen used in the preparation of the sample

I think we should be very careful in interpreting any form of accelerated soundness test, whether it be made on concrete or on aggregates In the first place, as everyone concerned with construction realizes, concrete is used under a wide variety of conditions of exposure As yet, we do not know the relation between these exposure conditions and the extreme conditions of exposure which obtain in our accelerated laboratory tests We know that certain types of materials will break down very badly when subjected to freezing and thawing, or to the sodium sulfate test and yet these same materials have been used for years with entire satisfaction The mere fact that they fail in some arbitrarily chosen accelerated laboratory test is not necessarily a proper criterion of their suitability for the service which they are supposed to render

Although some materials fail badly in an accelerated test, they seem not to cause any particular defect in concrete structures unless it be a slight blemish in the surface of the concrete after long years of exposure On the other hand, other materials which, in the laboratory test, fail no worse than the type of material which I have just mentioned, will cause concrete to expand and disintegrate in a comparatively short time when exposed to the weather Some materials expand with great force upon disintegration, others merely soften and break apart in the course of time One type of material is dangerous, the other type can be and has been used with perfect safety

The expansion tests reported by Mr Hanson are interesting and they indicate a type of investigation which should be extended in connection with weathering tests It will be noted that the expansion specimens designed for a six inch slump exhibited far greater expansion than those designed for a two inch slump, again suggesting that the free water, that is the uncombined water which creates porosity, had something to do with the amount of expansion obtained

A number of investigators have undertaken investigations in the weathering tests of aggregates and of concrete, each using his own particular method, but in the vast majority of cases the investigator is left in doubt as to the true meaning of his test in terms of the service value of the material under consideration Would it not be a fine thing if this matter of accelerated soundness testing could be tackled from a fundamental viewpoint by a number of investigators working together in a large cooperative investigation and including, in addition to laboratory tests, thorough inspections of the service behavior of the materials themselves? I venture to say that in the course of a very few years, were such a cooperative investigation outlined and carried forth, we should have a far more definite idea of the meaning of accelerated tests and the proper method of carrying them out than exists at the present writing Mr Hanson, I feel is working along the right lines and it is to be hoped that his work will be continued so that additional data will be accumulated He is to be congratulated on the character of the results which he has obtained They are not extensive but, none the less, they seem to be indicative and in agreement with one's good judgment Let me suggest that when the paper is published it will be made even more valuable by the inclusion of all of the essential data

REPORT OF COMMITTEE ON MAINTENANCE

B C TINEY, *Chairman*

TREATMENT OF ICY PAVEMENTS

SYNOPSIS

The reduction of traffic hazards caused by slippery roadways is one of the important problems demanding the attention of maintenance engineers at the present time and it is essential that a solution be found The formation of ice on the pavement surfaces constitutes one of the worst and most difficult conditions to combat It has been the practice of highway maintenance engineers of certain northern states to use calcium or sodium chloride for treatment of icy pavements These materials are usually added to sand or cinders and the mixture spread upon the travelled surface They serve to lower the freezing point and embed the abrasive particles in the ice, thus rendering the surface non-skid Considerable diversity of opinion, however, has existed concerning the most effective manner in which these chlorides should be used, whether or not they damage the surface of concrete pavements, and their relative ability to melt ice

Progress reports of methods in practice for treatment of icy pavements and investigations using concrete specimens of 28 days and six months of age have been presented in the Tenth, Eleventh and Twelfth Annual Proceedings of the Highway Research Board This report reviews the previous reports and presents the results of tests on concrete specimens one year of age, together with the recommendations of the Committee

For studying the values of calcium chloride and sodium chloride for the treatment of icy pavements a program of research on the following factors was outlined:

(1) Melting power of calcium chloride and sodium chloride at various low temperatures

(2) Minimum amount of each salt necessary to embed abrasive material in ice at various low temperatures

(3) Effects of calcium chloride and sodium chloride on the surfaces of concretes subjected to repeated freezings and thawings

(4) The value of pretreatment of the surface with various coverings to protect the concrete from possible scaling when these salts are used for thawing ice

Research in accordance with the above outline was conducted by the State Highway Department of Michigan and by the Engineer Department of the District of Columbia

Concrete blocks of approximately 100 square inches area were prepared with raised edges so that solutions of chlorides could be held on the surface Series of different consistencies of concrete, of different degrees of finish, and of different ages, using various concentrations of solution at various degrees of low temperature were included in the tests

Cement mortar specimens (2 x 4 inch cylinders) were prepared in two consistencies and two finishes. These cylinders were immersed to one-half their depth in solutions of sodium and calcium chloride

Both the block and cylinder specimens were subjected to 30 cycles of freezing and thawing.

In the report of the Michigan Department the data have been evaluated and graphs presented which show the results of the tests Figures 1 and 2 show clearly the relative effects of sodium and calcium chloride

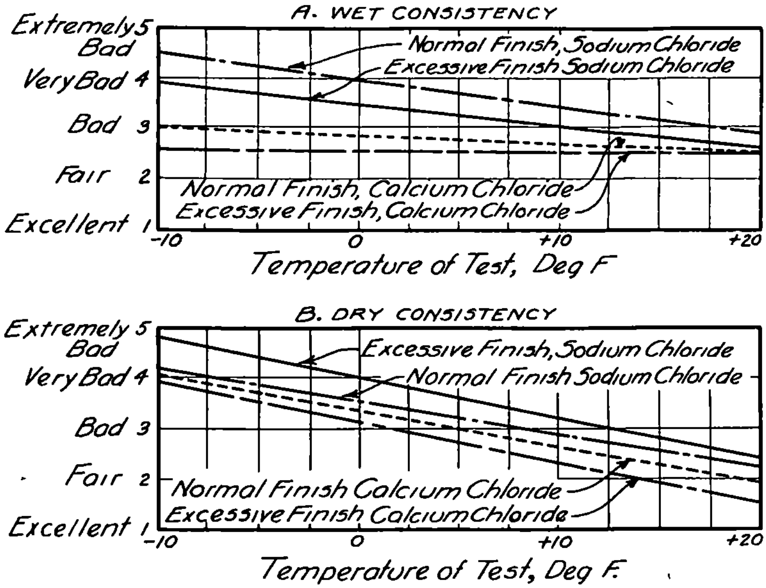

Figure 1. Surface Condition Rating of Specimens at End of 28 Days. Averages of Three Application Rates.

upon 28-day concrete of various consistencies and types of finish Figures 3 and 4 show this same comparison, but on specimens six months of age, taken from the results reported by the Engineer Department of the District of Columbia Figure 5 shows the relative effects of these chlorides on concrete specimens one year of age; taken from Michigan Department investigation

The program of tests carried out by the Engineer Department of the District of Columbia included some specimens covered with a 40 per cent solution of sodium silicate and some covered with 85–100 penetration asphalt previous to the application of the chloride solutions, as well as specimens having applications of various concentrations of chlorides and subjected to various temperatures Also specimens were included on which was applied a calcium chloride solution of such concentration that freezing did not occur

CONCLUSIONS

The relative ice-melting power of the two chlorides constitutes a measure of their ability to embed sand In these tests sodium chloride

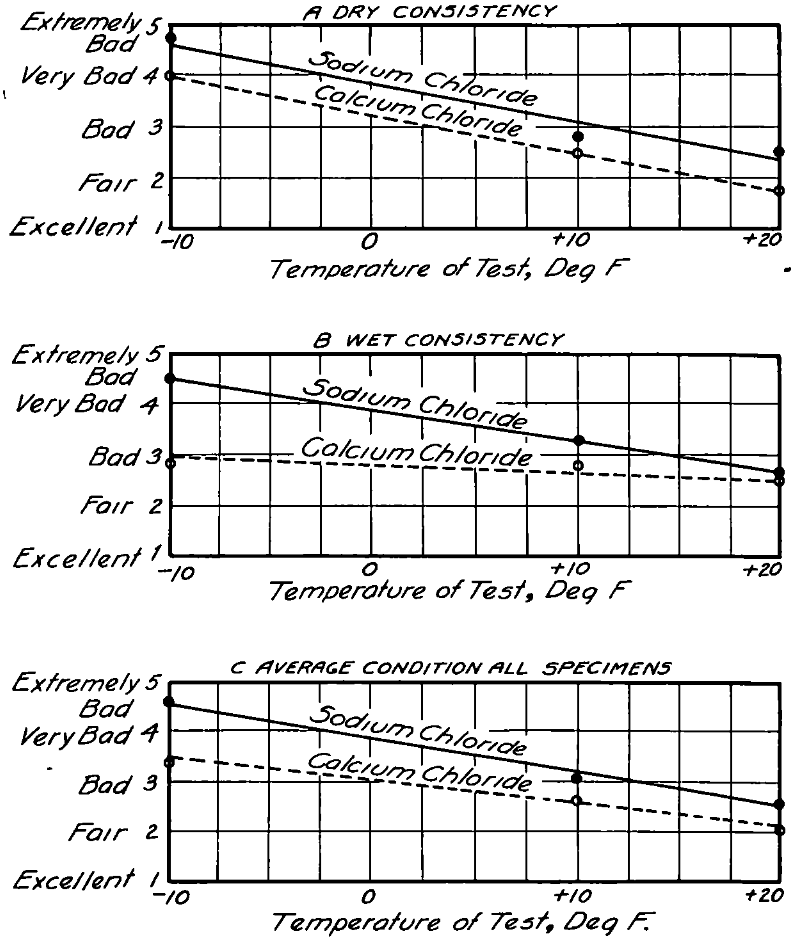

Figure 2 Surface Condition Rating of Specimens at End of 28 Days. Averages of Three Application Rates, Two Finishes, Two Consistencies

had the greater melting power and therefore the greater potential sand embedment property at temperatures above approximately plus 10°F Between plus 10°F and minus 6 5°F the differences in melting power are slight Below minus 6 5°F, approximately the eutectic point of sodium chloride, this salt has no melting or sand embedment power, whereas calcium chloride continues to be active down to its eutectic point, minus 58 5°F. Table I shows the relative melting capacities of

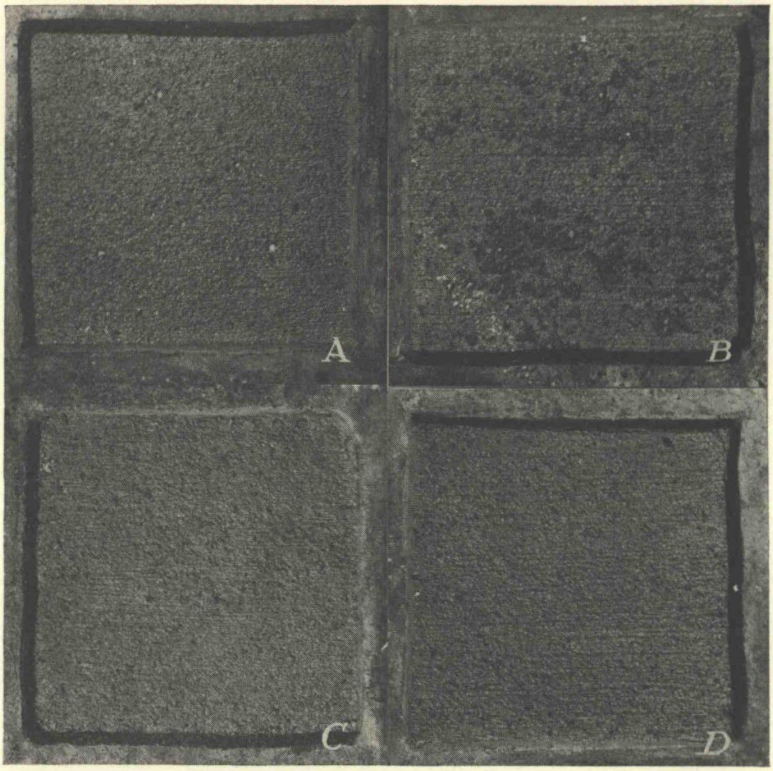

Figure 3. Effects of Calcium Chloride and Sodium Chloride Treatments on Surface of Concrete.

A. Four per cent calcium chloride on plain concrete (slight disintegration).
B. Four per cent sodium chloride on plain concrete (marked disintegration).
C. Eight per cent calcium chloride on plain concrete (slight disintegration).
D. Eight per cent sodium chloride on plain concrete (slight disintegration).

TABLE I

COMPARISON OF MELTING CAPACITIES OF CALCIUM AND SODIUM CHLORIDE

Laboratory Tests

Temperature Deg. F.	Pounds of Ice Melted Per Pound of Chemical	
	77-80 per cent Flake Calcium Chloride	Pure Sodium Chloride
−30°F.	2.9	—
−20	3.2	—
−10	3.5	—
−6.5	3.7	3.2
0	4.0	3.7
5	4.4	4.1
10	4.8	4.9
15	5.5	6.3
20	6.8	8.6
25	10.4	14.4
30	31.1	46.3

calcium and sodium chloride at various temperatures obtained in these tests.

Disintegration of concrete which may occur in connection with the treatment of icy pavements with chlorides, is not due to the action of the chlorides, but rather to the repeated freezing and thawing action produced by the treatment.

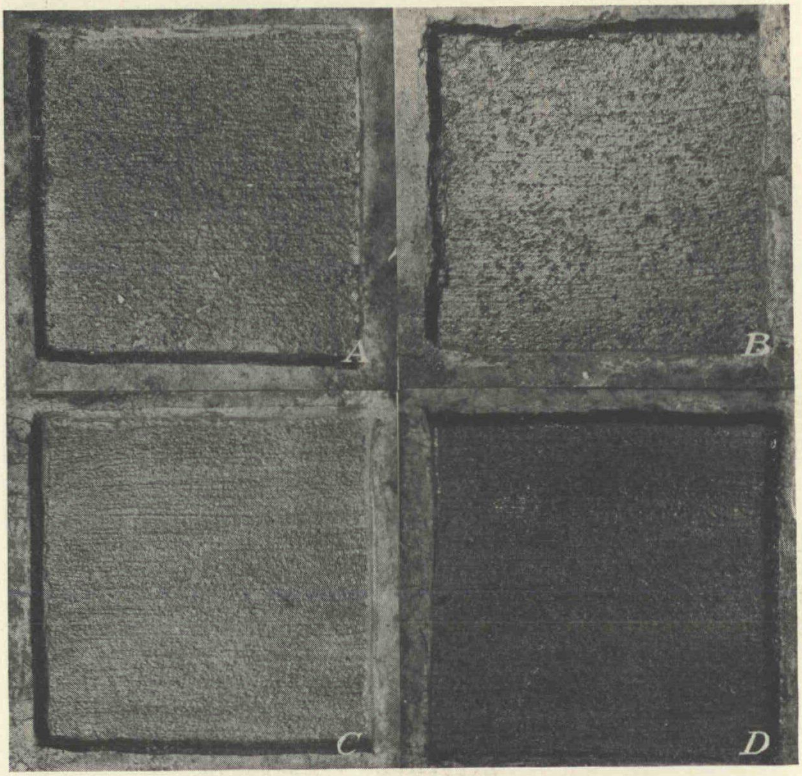

Figure 4. Effects of Treatments with Chlorides and Boiling Water upon Concrete Surface.

A. Twelve per cent calcium chloride on plain concrete (slight disintegration).

B. Twelve per cent sodium chloride on plain concrete (slight disintegration).

C. Thirty per cent calcium chloride on plain concrete (no disintegration).

D. Block from which an ice coating (plain water) was removed by the application of boiling water (slight disintegration).

Both sodium and calcium chloride, applied to the surface of concrete, increase the pitting and scaling which may occur from repeated freezing and thawing. The use of sodium chloride is more detrimental than the use of calcium chloride.

Mortar cylinders partially immersed in solutions of calcium and sodium chloride, and subjected to repeated freezing and thawing, showed damage to the mortar bond The sodium chloride was more detrimental than the calcium chloride since the specimens immersed in solutions of the former developed large longitudinal cracks with minor spider-web cracks, and in many cases the entire cylinders disintegrated Cylinders immersed in calcium chloride solutions developed only surface scaling

Coating of the concrete with oil, asphalt, or sodium silicate previous to making the tests did not prove of value in protecting the concrete against pitting or scaling of the surface

One of the most important factors controlling the severity of pitting and scaling is the amount of chert, shale, or soft stone that exists near the surface of the concrete It is evident from a detailed study of the specimens that the presence of such material near the surface would almost invariably result in deep pitting and removal of surface mortar

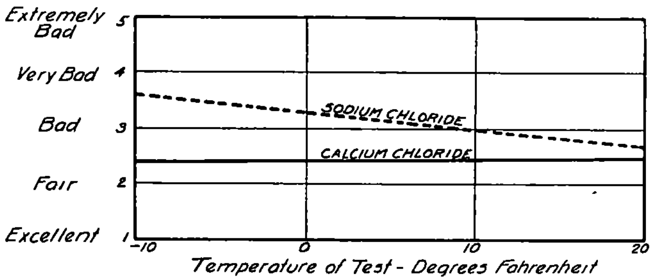

Figure 5. Surface Condition Rating of Specimens at Conclusion of Test Age One Year. Average Condition of All Specimens Three Application Rates, Two Consistencies, Two Finishes

It should be noted that the results from these tests were due to 30 cycles of freezing and thawing and the solutions of chlorides were the maximum amounts which might be used In practice, a considerable amount of the chloride would be absorbed or adsorbed by the abrasive and much of it would be dissipated by traffic Such results as are shown by these tests would be found only where concentrations of the solutions and abrasives were permitted to remain on the surface of the concrete and where conditions were such that repeated freezing and thawing would occur The tests do indicate, however, the importance of the proper supervision and care which should be exercised in using this method of treatment of icy pavements

Field observations of concrete pavements indicate that

There are comparatively few cases where pavement scaling can be attributed to the use of calcium chloride in the treatment of ice

Many pavements where calcium chloride has been used over a period of years in the treatment of ice, show no indications of scaling.

The Committee believes that the importance of safeguarding life and property on icy pavements far outweighs the minor damage that may be done to the pavements

RECOMMENDATIONS

After careful consideration of the results of these investigations, the Committee makes the following recommendations —

Use If the treatment is to be applied to cement concrete pavements, the use of calcium chloride is recommended.

In the case of pavements other than cement concrete the Committee has had no information that either calcium or sodium chloride will promote disintegration by freezing and thawing and therefore either of the chemicals may be used, excepting that calcium chloride is preferable at temperatures lower than 6 5°F below zero

Abrasive Materials Cinders are preferable, although coarse sand may be used since it is more generally available

Preparation of Stock Piles In treating stock piles to prevent freezing, 25 to 50 pounds of calcium chloride or sodium chloride for each cubic yard of abrasive should be spread over the piles either dry or in solution

Application At the time of application, the material should be treated with an additional fifty pounds of flake calcium chloride or sodium chloride per cubic yard The treated abrasive should be applied at the rate of 1½ to 2 pounds per square yard of pavement surface Mechanical spreading is preferable to hand work

The use of either chemical, without abrasives, for the purpose of removing ice is not recommended for general practice, excepting in cities where abrasives tend to clog drainage systems In such cases it is desirable to remove the resulting slush from the pavement

There is definite information that the presence of sand or cinders on the pavement surface, after the ice has melted, will cause damage to the pavement by grinding action under the wheels of vehicles It is therefore important not to apply an excess of sand or cinders and to remove any accumulations of such abrasives from the bottoms of grades or other locations where they may collect

DISCUSSION

ON

TREATMENT OF ICY PAVEMENTS

MR G A RAHN, *Pennsylvania Highway Department* The basic theory of embedding a gritty material in the ice, in order to provide tractive resistance for vehicular traffic, is sound

I note that the report recommends the use of flake calcium chloride, and does not mention calcium chloride solutions All of our work in Pennsylvania has been done by impregnating the cinders with a 35 per cent solution Due consideration was given to the various methods used with this material in conjunction with cinders, and in the final analysis it was felt that if the individual particles of the cinders could carry a certain portion of the salt, this would be much more efficient than a dry mixture of the flake and cinders, which at its best would tend to segregate

Our method consists of pouring the 35 per cent solution over the cinder pile immediately before use If a crust is formed this will disintegrate under the action of the solution The impregnated cinders are spread in a manner similar to dry cinders This will form a mat upon the icy surface, and an immediate reaction is noted, the particles being embedded to a depth of $\frac{1}{16}$ to $\frac{1}{2}$ inch, dependent on the density of the ice With untreated cinders it will require a patrol to spread them morning and evening, while the treated cinders remained efficient for as many as 14 days It is estimated that this makes a saving of between 65 and 70 per cent of the cost of cindering This method of treatment is not economical on ice which will only lay for a day or two, but is economical in sections where the icy condition is continuous over a considerable length of time

Flake calcium chloride has been found economical in the removal of ice from gutters, and will lower the cost of this operation approximately 50 per cent

Economics also enters into this problem from the standpoint that cinders are difficult to obtain, due to the shutdown and curtailed operation of industrial plants at the present time Even prior to the time which we are now passing through, it was ofttimes difficult to obtain cinders in the quantity necessary to carry on this work I think we all agree that cinder is the ideal material to use, and in the search for a substitute the inherent qualities of this material should be kept in mind In other words, my opinion is that the material should be black, gritty, and free from very fine particles.

Black, from the psychological standpoint that this material can be seen by the motorist, and will be conducive to a feeling of safety, which would be lacking in a light colored material I recall several instances in Pennsylvania when we received reports that certain hills had not been cindered Upon investigation we found a light colored granulated slag had been placed on the pavement, and due to its color it blended with the color of the ice, and in some cases had blown away This gives rise to another point, the material used should stay put A black material also absorbs heat, which aids in embedding the particle in the ice

Gritty, from the standpoint that it will be more readily embedded, gives tractive resistance and is not so easily removed under the action of traffic, as rounded material

Free from fines from the standpoint that an excess of fine material will produce a greasy condition which will tend to increase skidding rather than prevent it

The problem of making highways safe for traveling during all seasons of the year is one which cannot be passed by Winter travel, especially in the northern states, is hazardous at its best We should lend our best efforts in reducing this hazard to a minimum

DR C D LOOKER, *International Salt Company* I believe all of us agree that 30 cycles of freezing, thawing and drying out on successive days is a condition which is much more severe than is ever encountered in actual practice The drying out of a pavement after a sleet or snow storm is a much slower process By the time the ice has all melted and the road surface has dried, most of the chloride has been carried to the side of the road.

Practically all of the reports of injury to concrete have referred to railroad switches and ice cream stands where large amounts of salt (Sodium Chloride) had been used If calcium chloride had been used in like manner, the report proves that the concrete would, also, have been injured Sodium chloride is being used, correctly, either alone or in combination with sand or cinders by highway departments, transportation companies and cities without injury to concrete or other road surfaces.

Low Temperatures In the United States, except perhaps in the extreme north, there is very little occasion for treating pavements with sand or cinders which have been impregnated with chloride when the temperature is lower than minus 6 degrees Fahrenheit Table 1 and calculations from published works (Comptes Rendus 148, 550, 1909 and J Am Chem Soc 40, 1204, 1918) show that there is practically no difference in the melting power of sodium chloride and 77–80 per cent calcium chloride between 10 degrees F. and minus 6 degrees F

Neither chemical has appreciable sand embedment power at low temperatures As the temperature rises and the amount of traffic increases, sodium chloride will prove the more effective When the price of these two chemicals is considered, the savings are apparent when sodium chloride is used

Preparation of Stock Piles In order to make a mixture equivalent to the 35 per cent calcium chloride solution which has been recommended, dissolve 50 pounds of sodium chloride in 18 gallons of water This makes about 20 gallons of saturated brine Mix as much of this solution with the cinders as they will take up Spread 30 to 50 pounds of sodium chloride over each cubic yard of material before it is applied to the road at the rate of 1 5 to 2 pounds per square yard of surface Fine rock salt is best adapted to this purpose It may be easily dissolved if suspended in water near the top of the container in an open mesh bag or a basket made of screen wire

For those who do not care to use cinders or sand, a very thin layer of sodium chloride may be spread on the ice by means of a mechanical spreader

In the absence of field experiments to show that sodium chloride, when properly used, will not injure cement concrete pavements more than calcium chloride, the use of sodium chloride will prove economical and efficient on the thousands of miles of asphalt, tar-bound macadam, water-bound macadam, brick and gravel roads to make them safe for traffic

It is hoped that consideration will be given to the continuation of this research by some field work in which the effects of sodium chloride on concrete, when properly applied, be observed under actual traffic and highway conditions

REPORT OF SUBCOMMITTEE ON MAINTENANCE OF CONCRETE PAVEMENT CRACKS AND EXPANSION JOINTS

W H ROOT, *Subcommittee Chairman*

Maintenance Engineer, Iowa Highway Commission

SYNOPSIS

This report is a record of the conditions of application and results secured with various fillers on three experimental road sections in Iowa In 1931 five asphalts, three tars, one asphalt cut back and one asphalt emulsion were tried on one road In 1932 materials similar to the more successful asphalts and tars used in 1931 and a heavier cut back asphalt were used on two other roads

The cut back and the emulsion used in 1931 gave some trouble in application and neither appeared to solidify rapidly enough to be considered satisfactory for use under traffic The heavier emulsion gave satisfaction although it was difficult to apply

The asphalts with penetration around 100 at 25°C and fairly soft tars with a float test around 112 gave good results

The results of similar experiments in California and Connecticut are to be reported later

In 1931 this subcommittee of the Maintenance Committee of the Highway Research Board presented a statement of the results which are desired to be attained by the maintenance of expansion joints and cracks in concrete pavements This statement was as follows·

1 By the proper maintenance of cracks and joints in concrete pavements we hope to prolong the life of such pavements by
   a Reducing the spalling and breaking down of the concrete adjacent to the crack or joint, and
   b Deferring the blowing up of the pavement, due to compression failures and reducing the number of such failures

2 In order to maintain the cracks and joints adequately we should have a material which will:
   a Penetrate the entire depth of the crack,
   b Remain in the crack for several seasons,
   c Be stiff enough that it will not flow from the crown to the sides of the pavement and yet not so stiff that it will form objectionable ridges when forced out of the crack by expansion,
   d Pour cold, and
   e Largely prevent surface water and foreign material from entering the crack

TABLE 1

IDENTIFICATION OF CRACK FILLERS

Research Project R-70, On Iowa Primary Road No 15

Key	Brand of Material	Name of Producer
	Section A Asphalt Crack Fillers	
C	Genasco Crack Filler	Barber Asphalt Co
D	Genasco Block & Joint Filler	Barber Asphalt Co
Z	—	Carter-Waters Co
G	—	The Texas Co
H	—	Shell Petroleum Corp
	Section B Tar Crack Fillers	
A	Tarmac Hot Patch	Koppers Products Co
B	Tarmac Special Crack Filler	Koppers Products Co
F	Barrett Special Pitch	The Barrett Co
	Section C Asphalt Cut-Back	
I	—	Shell Petroleum Corp
	Section D Asphalt Emulsion	
E	Colas Quick Breaking Emulsion	The Flintkote Corp

At the meeting of the Maintenance Committee it was decided that three independent experimental projects should be carried on, one by Mr Dennis in California, one by Mr Hamlin in Connecticut and one by Mr Root, chairman of the subcommittee, in Iowa The Connecticut and California experiments are not complete and this report deals entirely with the Iowa project

In 1931 the Iowa Department chose Primary Road No 15 from Ames to the Hamilton county line for experimental study These 12 miles of road were constructed in 1931 with premolded fibre type expansion joints at intervals of 80 feet and at the time of the experiment no maintenance of cracks or joints had been done.

For the purpose of this investigation one drum samples of five asphalt crack fillers, three tar crack fillers, one asphalt cut-back and one emulsion crack filler were available The identification of these materials

TABLE II

CHARACTERISTICS OF ASPHALT CRACK FILLERS, SERIES OF 1931

Key	Lab No	Specific Gravity at 25°C	Softening Point (R & B) °C	Penetration 0°C 200 Gm 60 Sec	Penetration 25°C 100 Gm 5 Sec	Loss on Heating 5 Hrs @ 163°C %	Flash Point Deg F	Total Bitumen Soluble in CS_2 %	Ductility at 25°C Cm	Inorganic Ash %
C	B-462	1 118	46	29	91	0 4	460	87 2	100+	9 9
D	B-463	1 204	52	12	43	0 2	450	74 0	80	20 9
Z	B-464	1 005	44	34	130	0 4	565	99 7	100+	0 0
G	B-466	1 012	49	38	89	0 2	525	99 1	37	0 6
H	B-467	1 041	51	24	68	0 2	510	99 6	104	0 2

TABLE III

CHARACTERISTICS OF TAR CRACK FILLERS, SERIES OF 1931

Key	Lab No	Specific Gravity at 25°C	Float Test Seconds @ 50°C	Water %	Distillation Percentage by Weight Total to 170°C	Distillation Percentage by Weight Total to 270°C	Distillation Percentage by Weight Total to 300°C	Sp Gr of Distillate 0—300°C	Softening Pt of Residue (R & B) Deg C	Total Bitumen Soluble in CS_2 %	Inorganic Ash %
A	T-110	1 208	68	0 0	0 0	9 5	24 7	1 030	44 5	92 4	0 0
B	T-111	1 232	1800+	0 0	0 0	0 6	2 2	—	67	78 5	1 05
F	T-112	1 255	1800+	0 0	0 0	0 0	—	—	76	67 2	1 62

TABLE IV

CHARACTERISTICS OF ASPHALT CUT-BACK CRACK FILLER, SERIES OF 1931

Key	Lab No	Water %	Distillation Percentage by Volume Total to 225°C	Distillation Percentage by Volume Total to 315°C	Distillation Percentage by Volume Total to 360°C	Flash Point Deg F	Specific Viscosity (Engler) 50 cc, 50°C	Penetration of Residue 100 gm, 5 Sec @ 25°C	Total Bitumen in Residue Soluble in CS_2—%
I	B-468	Trace	0 0	22 5	30 0	190	63 7	Too Soft	99 0

TABLE V

CHARACTERISTICS OF ASPHALT EMULSION CRACK FILLER, SERIES OF 1931

Key	Lab No	Asphalt Content Percent	Penetration of Asphalt Residue 100 Gms, 5 Sec @ 25°C
E	B-465	57 0	120

is given in Table I The producer in each case furnished the material either from the producing plant or a company warehouse.

Shortly after the arrival of each drum of material at Ames, a sample was removed and filed in the Highway Commission Laboratory for testing The tests were performed after the completion of the field work, and the results are presented in Tables II, III, IV and V

A condensed record of the data recorded during the application of the fillers appears in Table VI.

TABLE VI

RECORDS OF APPLICATION OF CRACK FILLERS

On Primary Road No 15, North of Ames, Iowa

Construction Project No F-72B

Date of Filling	Filler Key	Location		Weather	Temperature Record Deg F			Heating Time Hours	Applying Time Hours
		From Station	To Station		Air	Pavement	Filler		
				Asphalt					
11/25/31	C	133	146	Fair	31 5	35 5	410	6 0	1 5
11/30/31		146	165	Cloudy	37 0	35 5	410	6 0	1 5
11/30/31		187	196	Cloudy	37 0	35 5	410		
11/30/31	D	165	187	Cloudy	37 0	32 5	400	4 5	2 5
11/25/31	Z	41	98	Fair	31 5	35 5	390	6 0	1 5
11/30/31	G	196	233	Cloudy	34 0	32 5	430	3 0	1 5
11/30/31	H	233	253	Cloudy	34 0	32 5	420	2 5	1 0
				Tar					
11/25/31	A	1	41	Fair	23 0	30 5	295	5 0	1 5
11/25/31	B	98	133	Fair	31 5	35 5	400	2 0	1 5
12/ 1/31	F	254	279	Fair	27 0	30 0	370	1 5	1 0
				Asphalt Cut-Back					
12/ 1/31	I	At Station 280		Fair	29 0	31 0	120	0 5	0 5
				Asphalt Emulsion					
12/ 1/31	E	At Station 282		Fair	34 0	37 0	—	—	0 5

PREPARATION AND HANDLING OF MATERIAL

The materials, excepting the asphalt emulsion, were prepared for use by heating in a small, portable, gas-heated kettle having a capacity of about one and one-half drums Each material was heated to the temperature at which it would readily flow from the pouring pots which were of the cone-shaped variety with bottom discharge For the asphalts a temperature in the neighborhood of 400°F was found high enough in most cases to cause the desired liquidity Temperatures for the tars were generally somewhat lower than that The asphalt cut-back could be heated safely to but little more than 120°F , at which temperature it was in condition to pour satisfactorily The asphalt emulsion flowed satisfactorily without heating, although its temperature was only slightly above that of the air or pavement

In Table IV, the temperature required to produce the optimum liquidity has been recorded for each material Record was also made of the time required for heating each charge The values varied principally on account of the differences in the sizes of the charges introduced into the kettle at the beginning of the melting The values shown for heating time are slight indication of the resistance of the material to melting

Satisfactory consistency for pouring was maintained throughout the entire charge of each material at the temperatures recorded, except that for asphalts C and D, which became steadily thicker in spite of the stirring given at frequent intervals during the heating and using of the batch. Slight increases of temperature offered but little remedy for this situation, which is believed to be due to the gradual segregation of the mineral matter contained in these two fillers Some form of mechanical agitation would be necessary to keep this material in suspension at the temperatures required for pouring these asphalts in cold weather.

BEHAVIOR OF FILLERS IMMEDIATELY AFTER PLACING

The hot material, when introduced into the cold crack, sometimes sputtered and boiled violently due to the presence of ice, ice particles, small pieces of frozen earth, or free water This action was usually of little consequence. Sometimes a small amount of frothy asphalt would overflow onto the pavement surface The subsidence of the material in the crack would then, in most cases, require the addition of more material to fill the crack The cold pavement soon cooled the material in the crack and the small amount unavoidably spilled on the edges in pouring.

During the application of all of these fillers the road was kept open to traffic The action of the freshly placed filler under traffic was noted for each material. Without exception, the asphalts and tars behaved satisfactorily, in that they were unaffected by the traffic None of these materials were picked up by the tires of cars or trucks, even immediately after they were poured.

The asphalt cut-back, however, was picked up extensively by the tires of passing cars; also it whipped out in strings and spattered the cars This material could be used only in case a rather heavy sanding was given the crack immediately after pouring This was tried and found to be a successful method of application, but of necessity a much slower process than that for fillers which could be used without sanding

The asphalt emulsion behaved similarly, except that it spattered more than the cut-back, and failed to respond satisfactorily to the sand treatment Neither the cut-back nor the emulsion appeared to solidify rapidly enough to be considered satisfactory crack fillers for use under traffic

These materials were observed frequently through the winter of 1931–1932 and a thorough inspection was made early in the summer of 1932 Tar A gave satisfactory results Tars B and F were too hard

TABLE VII

IDENTIFICATION OF CRACK FILLERS

Research Project R-70

On Primary Road No 14, Marshall County

Item	Section A Asphalt	Section B Tar	Section C Asphalt Cut-Back
Key	J	K	L
Brand of Material	Iowa State Highway Specification	Tarmac	Iowa State Highway Specification
Name of Producer	Pioneer Asphalt Co	Koppers Prod Co	Standard Oil Co

TABLE VIII

CHARACTERISTICS OF ASPHALT CRACK FILLERS, SERIES OF 1932

Key	Lab No	Specific Gravity at 25°C	Softening Point (R & B) Deg C	Penetration 0°C 200 Gm 60 Sec	Penetration 25°C 100 Gm 5 Sec	Loss on Heating 5 Hrs @ 163°C %	Flash Point Deg F	Total Bitumen Soluble in CS_2 %	Ductility at 25°C Cm	Inorganic Ash %
J	B-703	1 013	48	33	98	0 2	510	99 8	100+	—

TABLE IX

CHARACTERISTICS OF TAR CRACK FILLERS, SERIES OF 1932

Key	Lab No	Specific Gravity at 25°C	Float Test Seconds @ 50°C	Water %	Distillation Percent by Weight Total to 170°C	Distillation Percent by Weight Total to 270°C	Distillation Percent by Weight Total to 300°C	Sp Gr of Distillate 0—300°C	Softening Pt of Residue R & B Deg C	Total Bitumen Soluble in CS2 %	Inorganic Ash %
K	T-189	1 216	112	Nil	0 0	5 7	12 5	1 037	53	87 0	—

and were unsatisfactory Asphalt Z was satisfactory. Asphalts C, D, G and H seemed to lack adhesive qualities and were unsatisfactory Asphalt cut-back I and asphalt emulsion E were entirely unsatisfactory

In 1932 it was decided that this experiment should be carried further using an asphalt of the general characteristics of the 1931 Z asphalt and tar of the general characteristics of the 1931 A tar It was, also, decided that a heavier cut-back asphalt would be used in this experi-

TABLE X

CHARACTERISTICS OF ASPHALT CUT-BACK CRACK FILLERS, SERIES OF 1932

Key	Lab No	Water %	Distillation Percent by Volume Total to 225°C	315°C	360°C	Flash Deg F	Specific Viscosity (Engler) 50 cc, 50°C	Penetration of Residue 100 Gm, 5 Sec @ 25°C	Total Bitumen in Residue Soluble in CS_2 —%
L	B-704	Nil	11 0	20 0	77 5	80	90	82	99 8

TABLE XI

RECORDS OF APPLICATION OF CRACK FILLERS ON PRIMARY ROAD NO 14 NORTH OF MARSHALLTOWN, IOWA, AND ON PRIMARY ROAD NO 169, WEST AND NORTH OF OGDEN, IOWA

Material	Asphalt J		Tar K		Asphalt Cut-Back L	
Run Number	1	4	3	6	2	5
Location						
From Station	341+42	341+42	255+45	80+40	315+15	492+32
To Station	315+15	429+32	218+50	123+00	255+45	439+00
Date of Filling	11/21/32	11/21/32	11/23/32	11/28/32	11/21/32	11/23/32
Weather	Fair Quiet	Fair Quiet	Cloudy Windy	Fair Windy	Fair Quiet	Cloudy Windy
Temperature of Air, Deg F	15–26	22–32	27–28	27–32	28–32	26
Temperature of Pavement, Deg F	15–30	31–33	30	25–26	33	30
Temperature of Filler, Deg F	350–400	250–365	210–260	225	120–150	140
Heating Time, Hours	3 0	2 50	0 5	0 75	5 5	0 5
Applying Time, Hours	1 5	2 75	1 5	2 00	3 0	1 0
Number of Cracks Filled	34	147	49	76	83	16
Thickness of Expansion Joint, Inches	1 0	0 75	1 0	0 75	1 0	0 75
Spacing of Joints, Feet	80	60	80	60	80	60
Quantity of Filler Used, Lbs	400	400	525	525	428	65
Length of Section, Miles	0 497	1 665	0 700	0 807	1 130	0 183
Quantity of Filler per Mile, Lbs	805	240	750	651	379	355

Notes

1 Runs No 1 to 5, inclusive, were on Road No 14, and Run No 6 was on Road No 169

2 Cracks at joints in Run No 1 appeared to be generally a little more open than the others having one inch joint material

3 Cracks at joints in Run No 4 were somewhat smaller than those of the other runs for this thickness of joint material

4 Runs Nos 1, 2 and 3 were on limestone concrete, and Nos 4, 5 and 6 on gravel concrete

5 All pavement cured during construction with 24 hours of wet burlap, followed by asphalt emulsion coating, which was in turn coated with white-wash

ment A newly constructed pavement on Road No. 14 north of Marshalltown was chosen, also, a section of Road No 169 west and north of Ogden These pavements were constructed in 1932, some with one inch joints spaced at 80 foot intervals and some with $\frac{3}{4}$ inch joints spaced at 60 foot intervals This information is shown in Table XI In this experiment the joints which had opened were filled and also all cracks which had formed between the joints were filled Table VII shows the identification of the crack fillers used, Tables VIII, IX, and X the characteristics of the materials and Table XI general information The tar and asphalt were applied in the same manner as described in the 1931 experiment

The asphalt cut-back used in the series of 1932 does not appear to flow freely at any temperature to which it is safe to heat it in open kettles With care it may be handled at temperatures between 120 and 140°F. However, as it has a flash point of 80°F , great care must be used to prevent fire It flows sluggishly into the crack and frequently shows considerable settlement in cracks that were full when poured It sticks to wheels of passing traffic slightly when warm, but does not appear to adhere to them after cooling It sticks fairly well to concrete, and remains soft at 30°F when freshly placed

A year after the application of these materials a careful inspection of the roads was made and all three seemed to have given good satisfaction although it will be noted from the above that the asphalt cut-back was very difficult to apply

CONCLUSIONS

From these experiments it appears that asphalt of the general character used in the 1932 experiment with a penetration of about 100 at 25°C or fairly soft tar with a float test of about 112 makes satisfactory crack fillers. We are almost convinced, however, that the filling of openings adjacent to premolded expansion joints is a waste of material This filler, together with a considerable portion of the expansion joint material, is forced out of the crack by the expansion of the pavement during the first warm days in the spring and we have found it necessary to cut this excess material from the surface of the pavement with a motor grader As long as there is a fair thickness of expansion joint material in the joint we believe that foreign particles entering the crack along the side of the joint will not cause spalling but will be embedded in the expansion joint material

DISCUSSION

ON

MAINTENANCE OF CRACKS AND JOINTS

MR B FREIBERG, *Laclede Steel Co* Has any research been done in Iowa to find out whether material comes up into the crack and expansion joint from below or whether it comes primarily from the top? In other words, when we attempt to seal expansion or contraction joints, is it more necessary to seal them from below or from the top?

MR ROOT: We have never made any experiments along that line

CHAIRMAN LANG: I take it from Mr Root's paper what he has in mind is spalling—that the filling of the crack has not been of any particular aid in preventing spalling A number of states are having trouble with what they call high joints, which appears to be due in some places to water getting into the joints causing swelling of the soil and in other places it is a distinct formation of ice crystals or layers

REPORT OF COMMITTEE ON TRAFFIC

W A Van Duzer, *Chairman*

Director of Vehicles and Traffic, District of Columbia

MOTOR VEHICLE ACCIDENTS AS REFLECTED BY PSYCHOLOGICAL TESTS AND REACTION METER

By W A Van Duzer

SYNOPSIS

Accident records show that the majority of traffic accidents are caused by a small percentage of the drivers It is desirable to devise some method to determine what drivers are likely to have accidents Applicants for drivers' licenses in Washington are required to answer orally a number of questions concerning the regulations Recently, some 200 applicants were given an additional written psychological test consisting of twenty-four questions and a reaction time test The applicants were graded and their ratings have been compared with their accident records with indications that it can be forecast which drivers will have the worst accident records It is recommended that applicants for licenses be given a written psychological test, a physical examination, a driving test, and, if it can be devised a test to measure emotional stability

There are 27 States, including the District of Columbia, which compel motor vehicle operators to have a license This means that 22 States do not require a driver's permit Some States require written examination, others written and oral, some oral and some no examination at all The majority of States, however, require a driver's test The minimum age limit is 12 years, in South Carolina, and several States have 14, 15 and 16 years The methods of examining the drivers and the questions asked are almost as numerous as the number of States listed

Opinions as the requirements of a safe operator are plentiful, but the actual measurement of the applicant's ability for the operation of a motor vehicle is very inadequate The majority of the States have only two points in common—that is, they are all insufficient and unreliable It seems to me that if driving ability really exists it can be measured and tests prepared that will obtain this measurement

A motor vehicle administrator has only one desire and that is to keep reckless and accident-prone drivers from operating motor vehicles This can be done by education A large proportion of all accidents is caused by a group of drivers composed of a relatively small percentage of the total number Therefore, the first concern of those in charge of motor vehicle operation should be the detection and the elimination of these incompetent drivers before, and not after, they obtain their driver's permits

The highway engineer has contributed much in making the streets and highways safe for vehicles operated in a reasonable and safe manner He has eliminated the more dangerous curves, widened and super-elevated curves on highways in rural districts, widened streets in urban sections, reduced grade crossings, and has generally increased road capacity

The automotive engineer has likewise contributed to making our streets and highways safe Most cars are now equipped with four-wheel brakes, safety glass, steel frames, balloon tires and tire treads that practically eliminate old-fashioned skid chains There has been improvement in lighting during the past few years, but the lights in use leave a great deal yet to be desired We believe also that much can yet be done by both the highway and the automotive engineer toward making the vehicle and the road safe for motor vehicle operation

Approximately 75 per cent of automobile accidents are attributed to the drivers We think human engineers—if we may use that term—should now contribute in a greater measure toward the reduction of accidents. Motor vehicle administrators must, with the help of the legislatures if necessary, put into effect laws and regulations which will eliminate the incompetent drivers by education, if possible, or examinations which will restrict the incompetent or careless operator

The District of Columbia has about the average system for licensing drivers, but we realize that we are not putting into effect the restrictions on the issuance of operator's licenses that it is possible to make

With this thought in mind we had all the applicants for drivers' permits on May 18th, 19th and 20th, of 1933, take a short written test to see if a test of this type would show any correlation with traffic violations We made this experiment in an effort to find out if a short, written psychological test would help in picking out the good driver from the score he had made in the test While it may be a little early to make any predictions the results seem gratifying

We gave the test to 300 persons on the days mentioned, but our study is based on the 210 of them who got their permits to drive The rest never received their permits This shows that our group was a selected one to begin with.

The test itself is composed of three parts. Part I has twenty-four True-False questions dealing with driving situations, Part II has five completion questions dealing with the driving regulations, Part III has nine multiple choice questions, designed to test driving judgment

In order to check their records we put the drivers' permit numbers, along with their test scores in the files at the Traffic Bureau A check was made after three months and another at six months Oddly enough the number of applicants who were arrested in the three-month period just about doubled when the six-month period was checked, and two persons got one violation in each period

Out of the 210 persons tested and checked, 26 were found to have records at the end of the six-month period Seven persons had two violations and two persons had three violations Only six applicants out of the 26 who had violations were above the average score and only two of the six had scores above 75 per cent

Looking at the test from a statistical standpoint we found we had a range of scores from six, as the lowest, to 48, as the highest, with the average at 36 The average was a little higher than we expected, but we believe it is due to the selection, since we took only the scores of the successful applicants We had eliminated about 40 per cent who did not get their permits, therefore, we might expect the average to be a little higher than it would have been had we based it on the total number who applied

While it is recognized that a test of this type would have to be put in alternate forms, say four or five, and the forms would have to be worked out on a much larger number of cases, it does seem like a step in the right direction

It is generally agreed that an actual operating test is essential to determine the ability of the applicant to drive a car, and this should be more than merely driving around the block and parking the car The time has come when we must realize that driving an automobile is a privilege and not a right, and the sooner we become more strict in the licensing of our drivers the better

In closing I recommend the following for the licensing of drivers.

(1) A written psychological test
(2) A physical examination of applicants and a periodic one of drivers in the future
(3) An eye test with color and peripheral test
(4) An actual driving demonstration
(5) Some test for measuring the emotional stability of the individual

Mr Roy Brown, of the Firestone Rubber Company, has built a machine which measures in one hundredths of a second, first, the time elapsing from the instant a light appears to the time that the applicant removes his foot from the pedal corresponding to the accelerator and second, the time from the instant that a light flashes that he takes to move his foot from the pedal corresponding to the accelerator to the pedal corresponding to the brake and to start the application of the brake

This machine has been used by the Psychological Laboratory at George Washington University and also by the Department of Vehicles and Traffic of the District of Columbia The Department of Vehicles and Traffic gave some 300 subjects this test, but sufficient time has not elapsed to make a thorough analysis of the data However, a hurried inspection indicates the following

1 The average reaction time under No 1—that is, the time elapsing from the instant the light goes on to the time the applicant takes his foot off the accelerator—is about 0 1 per second Under No 2, or the time elapsing from the instant the light goes on and the applicant takes his foot off the accelerator and starts to apply the brake it is about 0 2 of a second

2 Within certain limits (16 to 55) chronological age is not very closely related to reaction time. At the upper end of the age scale, however, Dr Walter R Miles, Professor of Psychology, Yale University, has made tests which seem to indicate that the reaction time slows up considerably.

Unquestionably the reaction of the subject from the time he is given a signal to the time the brake is applied is a factor in an increase or reduction of motor vehicle accidents If a motor vehicle is going at a rate of 40 miles an hour and the reaction is 0 3 of a second longer than it should be a car will move approximately 20 feet during this period This may be sufficient to cause a serious accident

NOTES ON TRAFFIC SPEEDS

By A N Johnson

Dean, College of Engineering, University of Maryland

SYNOPSIS

During the summer of 1933 the Maryland State Roads Commission in cooperation with the University of Maryland carried on a highway traffic speed survey to obtain comprehensive knowledge of the way traffic actually uses the state highways

The speed of traffic was observed at about 50 of the regular traffic census stations which the State Roads Commission has used for many years

At each point two observers counted and measured the speed of traffic from 9 A M until 4 P M This was done by the use of the Eno Foundation speed detector as devised by Professor C J Tilden of Yale University About 500 vehicles were timed in each direction at each station

The average speed as observed from 41,000 vehicles was 35 5 miles per hour, with 87 per cent of all the traffic within 45 miles per hour and 99 per cent within 55 miles per hour and with only an occasional vehicle moving over 65 miles per hour

The percentage of various rate of speeds was

8 per cent between 15–25 miles per hour
36 per cent between 25–35 miles per hour
43 per cent between 35–45 miles per hour
12 per cent between 45–55 miles per hour
1 per cent between 55–65 miles per hour

During the summer of 1933, there was undertaken by the Maryland State Roads Commission, in cooperation with the University of Maryland, a traffic speed census. The object was to ascertain the speeds at which highway traffic moves over the state highways

For this purpose, traffic speeds were observed at 54 stations, as shown on the sketch in Figure 1 At each point selected, which corresponded to some regular traffic census station, two observers measured speeds from 9 A M until 4 P M One observer counted the traffic, divided as to direction, while the other observed the speed It was the plan to measure the speed of 1000 vehicles, 500 in each direction at each station At a number of stations, however, the traffic proved to be insufficient to make 1000 observations in the time available

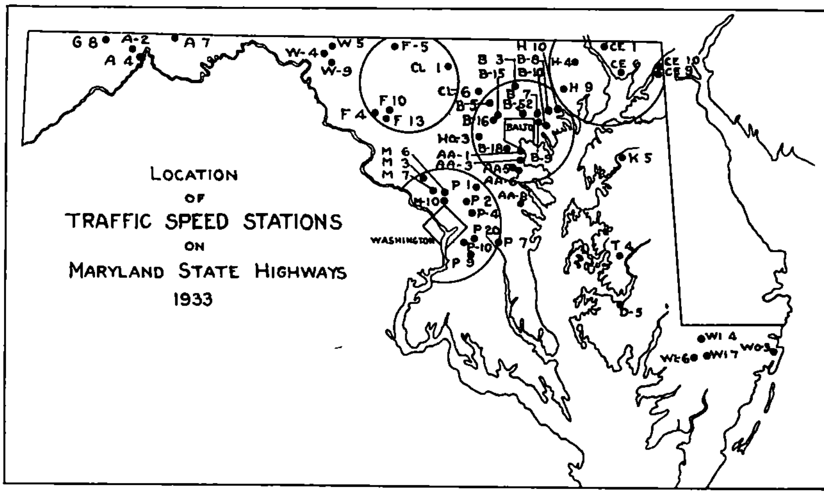

Fig. 1

The method of measuring the speed was by the aid of an Eno Speed Detector, as developed by Professor C J Tilden of Yale University It is a very simple device which consists of a box with two sides open On the diagonal plane is placed a mirror The observer stands 176 feet from the mirror, which is placed so that objects moving along the road will be reflected to the observer's eye He notes the time on a stop watch as a vehicle goes by him and again as it is flashed in the mirror Thus, if this time is two seconds, it indicates a speed of 60 miles per hour, three seconds, a speed of 40 miles, and so on

This method of determining the speed of vehicles was checked several times by comparing the speed thus recorded with that indicated by state police on their speedometers No practical difference was observed

It was possible to carry on this work without attracting undue attention of motorists, in fact, but few of the drivers noticed it at all, and those that did had slowed up after the speed had been observed

The highest average speed for any of the 54 stations was 47 miles per hour, with an average of 49 miles per hour for the east bound traffic, and 45 miles per hour for the west bound traffic This was on the Elkton-Glasgow Road, Route U S 40 There were six stations at which the average speed observed was over 40 miles per hour

The majority of the stations occupied for observing speed were in the zone where the legal limit is 40 miles per hour, the average speed in this zone being 37 miles per hour (See Figure 2, also Table I)

A few stations were occupied in more restricted zones In the 35-mile zones, the average speed observed was 33 miles per hour

In the 25-mile zone, where traffic was observed at four stations, the average speed was 34 miles per hour

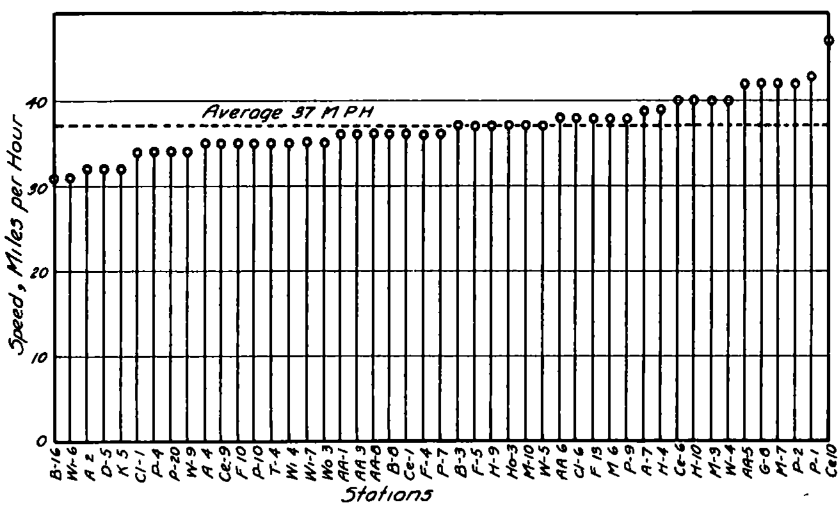

Figure 2 Range of Average Traffic Speed at Various Stations in 40 m p h Zone on the Maryland State Highway System Each speed is the result of several hundred observations See Table 1.

The following table shows the distribution of traffic according to speed in the 40-mile zone

10 per cent of all the traffic was within 15–25 miles per hour
32 per cent of all the traffic was within 25-35 miles per hour
45 per cent of all the traffic was within 35–45 miles per hour
11 per cent of all the traffic was within 45–55 miles per hour
2 per cent of all the traffic was within 55–65 miles per hour

It is seen that 87 per cent of the traffic was under 45 miles per hour, and but two per cent was over 55 miles per hour

Of the 52,704 cars timed, there were only 90 traveling faster than 60 miles per hour, divided as follows:

19 cars at 63 miles per hour
55 cars at 66 miles per hour
5 cars at 70 miles per hour
10 cars at 75 miles per hour
1 car at 80 miles per hour

TABLE I

Maryland Traffic Speed Survey, 1933

Road and Route	Traffic Station Number	Av No Vehicles Hourly	Av Speed Vehicles Both Directions M P H	Percentage of Traffic at Speeds Shown: Av Speed N/S	Av Speed E/W	15–25 N/S	15–25 E/W	25–35 N/S	25–35 E/W	35–45 N/S	35–45 E/W	45–55 N/S	45–55 E/W	55–65 N/S	55–65 E/W	More than 65 N/S	More than 65 E/W	No of Vehicles Timed
40 Mile Zone																		
Cumberland–Frostburg, No 40	A-2	200	32		34 30		6 21		45 51		41 26		8 2					1,073
Cumberland–McCool, No 220	A-4	175	35	34 36		6 5		43 40		46 45		5 9		1				1,030
Cumberland–Hancock, No 40	A-7	76	39		38 40		2 5		25 21		52 43		20 23		1 5		1	530
Annapolis Boulevard, No 2	AA-1	345	36	37 35		3 2		45 35		45 31		7 11		1				1,382
Annapolis Boulevard, No 2	AA-3	233	36	35 37		6 4		33 26		49 55		12 14		1				1,267
Annapolis Boulevard, No 2	AA-6	153	38	36 40		5 2		28 18		54 52		12 25		1 2				1,046
Crain Highway, No 3	AA-5	166	42	40 44		3 2		15 11		52 36		27 41		3 9		1		1,134
Defense Highway, No 50	AA-8	152	36		38 34		6 7		28 39		48 48		16 6		2			921

York Road, No 111	B-3	108	37	37 37		4 3		27 32		51 52		16 12		1 1		1		764
Belair Road, No 1	B-8	230	36	35 37		5 3		36 32		48 51		10 13		1 1				1,149
Liberty Road, No 26	B-16	123	31		32 30		21 26		45 48		29 23		5 3					597
Conowingo–Rising Sun, No 1	CE-1	140	36		35 37		14 4		28 25		47 54		10 16		1 1			905
Philadelphia Road, No 40	CE-6	194	40	38 42		5 1		26 4		52 47		16 32		1 6				1,075
Elkton–Chesapeake City, No 213	CE-9	70	35	35 35		9 9		34 30		46 46		11 12		 2		 1		537
Elkton–Glasgow, No 40	CE-10	190	47		49 45		1 1		7 8		24 44		43 40		21 6		4 1	1,133
Westminster–Frizzellburg, No 32	CL-1	90	34		35 33		7 11		36 46		45 38		12 5					676
Westminster–Reistertown, No 140	CL-6	116	38		38 38		3 4		23 25		55 50		18 20		1 1			880
Cambridge–Jacktown, No 16	D-5	194	32		31 33		18 15		50 41		30 35		2 9					1,019
Frederick–Harpers Ferry, No 340	F-4	97	36	37 33		6 10		32 50		46 36		13 4		3				865
Emmitsburg–Frederick, No 15	F-5	80	37	37 37		4 13		31 50		46 35		17 2		2				535

TABLE I—*Continued*

Road and Route	Traffic Station Number	Av No Vehicles Hourly	Av Speed Vehicles Both Directions M P H	Percentage of Traffic at Speeds Shown														No of Vehicles Timed
				Av Speed		15-25		25-35		35-45		45-55		55-65		More than 65		
				N/S	E/W	N/S	E/W	N/S	E/W	N/S	E/W	N/S	E/W	N/S	E/W	N/S	E/W	
40 Mile Zone—Continued																		
Frederick-Baltimore, No 40	F-10	137	35		35		8		38		44		9		1			903
					35		9		38		43		9		1			
Frederick-Rockville, No 240	F-13	145	38	38		4		26		54		14		1		1		857
				38		3		30		48		16		3				
Frostburg-Grantsville, No 40	G-8	69	42		42		3		13		42		38		4			525
					42		5		15		40		32		8			
Belair-Conowingo, No 1	H-4	136	39	37		3		30		54		12		1				891
				41		2		16		52		27		3				
Belair-Baltimore, No 1	H-9	266	37	37		3		31		53		12		1				1,282
				37		4		29		55		12						
Philadelphia Road, No 40	H-10	242	40	42		1		13		52		29		5				1,076
				38		4		26		53		16		1				
Frederick Road, No 40	Ho-3	127	37	38		6		33		48		12		1				1,355
				36		5		27		50		16		2				
Chestertown-Church Hill, No 213	K-5	65	32	33		15		43		32		10						507
				31		20		42		34		5						

Rockville–Frederick, No 240	M-3	161	40	41 39		2 2		18 21		51 53		24 21		5 3				1,019
Norbeck–Sligo, No 97	M-6	115	38	39 37		5 5		21 27		53 52		19 15		2 1				808
Rockville–Washington, No 240	M-7	203	42	42 47		1 1		16 13		49 49		29 32		5 5				1,223
East–West Highway, No 410	M-10	200	37		36 38		3 2		33 23		55 54		8 18		1 3			1,233
Baltimore–Washington, No 1	P-1	517	43	42 44		1 1		15 10		50 38		29 42		4 9				1,865
Baltimore–Washington, No 1	P-2	665	42	42 40				12 18		52 54		32 25		4 3				1,771
Defense Highway, No 50	P-4	174	34		36 32		6 15		36 47		51 35		6 3					903
Crain Highway, No 4	P-7	78	36		38 34		2 9		29 42		50 43		17 5		2 1			591
Camp Spring-T B, No 5	P-9	74	38		39 37		5 3		18 36		53 47		22 13		2 1			520
Camp Spring-D C Line, No 5	P-10	118	35		36 34		11 10		32 36		47 47		10 7					779
Marlboro Pike, No 4	P-20	135	34		35 33		5 9		40 48		44 38		11 5					818
Easton–Preston, No 213	T-4	85	35		35 35		9 13		32 48		48 36		11 3					545

TABLE I—*Concluded*

Road and Route	Traffic Station Number	Av No Vehicles Hourly	Av Speed Vehicles Both Directions M P H	Percentage of Traffic at Speeds Shown														No of Vehicles Timed
				Av Speed		15–25		25–35		35–45		45–55		55–65		More than 65		
				N S	E W	N S	E W	N S	E W	N S	E W	N S	E W	N S	E W	N S	E W	
40 Mile Zone—Concluded																		
Hagerstown–Hancock, No 40	W-4	140	40		42 38		1 3		18 25		49 49		26 21		4 2		2	1,031
Hagerstown–Middleburg, No 11	W-5	121	37	37 38		5 4		31 26		46 46		17 21		1 2		1		1,008
Hagerstown–Frederick, No 40	W-9	190	34		36 32		4 7		34 51		48 38		13 4		1			1,087
Delmar–Salisbury, No. 13	W1-4	162	35	33 37		9 6		51 32		36 48		4 13		1				1,042
Salisbury–Princess Anne, No 13	W1-6	129	31	31 31		17 15		57 58		25 25		1 2						1,312
Salisbury–Berlin, No 213	W1-7	118	35		34 36		12 10		49 37		36 41		3 11		1			918
Berlin–Ocean City, No 213	Wo-3	137	35		34 36		11 5		42 39		42 41		5 15					961
Average			37				10		32		45		11		2			Total 45,348

					Less than 15	15–25	25–35	35–45	45–55	55–65	
35 Mile Zone											
Belair Road, No 1	B-9	325	32	32 32	7 7	54 53	36 38	3 2			1,044
York Road, No 111	B-52	428	34	35 33	6 7	37 51	46 40	10 2	1		1,152
Average			33		7	49	40	4			
25 Mile Zone											
Reistertown Road, No 140	B-5	222	36	33 39	14 1	45 23	39 55	7 8	 3		873
Reistertown Road, No 140	B-15	350	34	34 34	6 4	49 48	42 45	5 3			1,040
Philadelphia Road, No 40	B-10	308	32	33 31	10 12	46 55	41 39	3 2			1,069
Baltimore–Washington, No. 1	B-18	716	35	36 34	2 2	31 56	58 45	9 3			1,442
Average			34		6	43	44	6	1		
20 Mile Zone											
Harford Road, No 147	B-7	223	26	27 25	1	38 21	51 59	10 19	 1		736
Average					1	29	55	14	1		

The car traveling at 80 miles per hour was on the Elkton-Glasgow Road, two miles east of Elkton

On three roads it happened that counts were taken on the same day as the county fairs and the speed records show that the cars traveling towards these events were going at a much higher rate of speed than the cars going in the opposite direction

Thus, at station AA-5 on the Crain Highway, the average speed of the traffic during the rush to the races was 48 miles per hour, while the speed of traffic in the opposite direction was but 39 miles per hour

In all, of the 52,704 vehicles that were timed, 45,348 were at stations within the 40-mile zone, and the remainder were in the lower speed zones

From the results obtained, it appears that 40 to 45 miles per hour is a reasonable regulation, which is observed by the large majority of drivers in all parts of the state The driving public on our highways is content for the most part to jog along at a moderate rate of speed

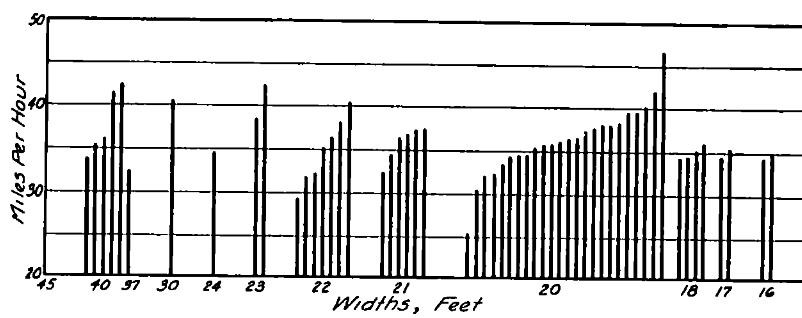

Figure 3. Relation Between Width of Pavement and Speed of Traffic

The relation that the width of pavement has upon speed of traffic was studied but without convincing results

For this purpose the data were assembled in Figure 3 showing the average speed at the different stations, arranged according to width of road

In this figure, each of the ordinates represents the mean of observations taken at a single station, averaging about one thousand vehicles per station It will be noted that the average for each of the different widths is not far from the same value, about 35 or 36 miles per hour

One thing that is notable is that the 18, 17 and 16-foot widths, while maintaining the same average speed as the wider widths of roadway, do not show as great a variation It is evident that more data than are here presented will be required to determine definitely what effect the width of the roadway has upon the speed of traffic

DISCUSSION

ON

HIGHWAY TRAFFIC SPEEDS

Captain L. A. Lyon, *Deputy Superintendent, Uniform Division, Department of Public Safety, Michigan:* This Department has recently completed a survey of the speeds at which motor vehicles are being driven on the Michigan highways. This survey was made by all of the officers of the Department while on their regular patrols, either with motorcycle or patrol car. The checks were made on all types of highways and under various weather conditions, from the congested area around Detroit to the lonesome trails of the far north.

The officers were instructed to pace the first ten vehicles they could on the various types of roads each day for two weeks. Thus, they did not pick out passenger cars or high speed vehicles. They were also instructed to make the check without the knowledge of the driver of the vehicle, if possible.

Cars were checked under the following conditions:

47 per cent during daylight.
53 per cent during darkness.
16 per cent during rain or on wet roads.
10 per cent on gravel roads, and the balance on various types of hard surfaced roads, mostly concrete.

The percentages of the vehicles traveling at the various speeds were as follows:

25 miles per hour and under	5
30 miles per hour	9.7
35 miles per hour	12.6
40 miles per hour	20
45 miles per hour	17.9
50 miles per hour	16.6
55 miles per hour	8.6
60 miles per hour and over	9.6

Little difference in average speed can be noted as between wet and dry roads.

On paved highways the average speeds were: during daylight, 43.3 miles per hour; during darkness, 41.5 miles per hour.

Two per cent of all vehicles checked on pavements in daylight hours were trucks; 1.5 per cent during night hours were trucks.

On gravel highways the average speeds were: during daylight 39.8 miles per hour; during darkness 34.0 miles per hour.

On gravel roads 4.5 per cent of all vehicles checked in daylight hours were trucks; during darkness 3.7 per cent were trucks.

Sixteen per cent of all cars checked were of foreign registration and were probably through traffic, and nearly all on pavement. The average speeds were during daylight, 48 1 miles per hour, during darkness, 47 2 miles per hour.

The cars that were apparently from some other part of the state and could be classed as through traffic constituted 22 7 per cent The average speeds were: during daylight, 48 3 miles per hour; during darkness, 48 miles per hour.

We find that the average speed of all vehicles checked on all types of highways and under all weather conditions is 43 7 miles per hour

The above data are all we can obtain from the reports as submitted With this experience we expect to make a more detailed survey in the coming year, and would like suggestions as to data that should be included

The results of the survey seem to indicate that 45 miles per hour is a good average speed, and that a regulation declaring speeds in excess of that figure to be prima facie evidence of unreasonable speed would be in order

ALCOHOL AND MOTOR VEHICLE DRIVERS

By Dr W R Miles

Yale Institute of Human Relations

SYNOPSIS

A man may keep his car right side up and on the road when he is too intoxicated to walk but this fact is not reassuring to others on the highway Although beverage alcohol appears to give subjective stimulant action to a person, its real effect is a depressant action on most of the functions of body and mind The alcohol effect which interferes with driving ability is fourfold. (1) A poorer grade of attention to external signals and environment, (2) Slower responses of eyes, hands and feet, (3) Less dependable, that is more variable, muscular responses, (4) Increased self assurance which prompts to the assumption of right-of-way and willingness to take a chance Although alcohol is directly mentioned in only 7 to 10 per cent of fatal highway traffic accidents, it is the belief of informed traffic officials that one-third of such accidents are at least partly chargeable to use of alcohol by the driver Officials need a method to definitely determine whether a driver is intoxicated as a basis for court action Determination of percentage of alcohol in the blood or urine by biochemical means is a feasible undertaking This method should be tried out in some representative areas to secure scientific data in this controversial field

The motor car of today is a splendid mechanism, comfortable, responsive and powerful, a truly marvelous extension of the human personality but it can not drive itself, society counts on its being used by responsible people who have reached and are maintaining the human adult level of understanding and emotional balance A large army of engineers is continually at work improving the automobile in all possible ways consistent with reasonable economy: for example in giving it greater pickup and speed plus effective traction and brakes Another army is equally energetic and effective in providing roads and highways suitable in special pattern and in surface characteristics for motor traffic The human factor is the third dimension in this picture and it is the most difficult one to treat successfully The third army, therefore, needs to be and is the largest of the three It has been relatively slow in mobilizing and has found difficulty in analyzing its field and organizing its program The task of the human engineer is comparable to that of the automotive engineer or the highway engineer, for in the last analysis, he is interested in designing, constructing, testing, remodelling, etc but here the materials worked upon are human habits, psychological attitudes, economic stresses, and social customs Within this special field of human engineering, traffic accidents and fatalities constitute the all too literal "break-down-test" from which crucial facts applicable to general practise are derived And within the traffic accident division of the field falls the subject of alcohol intoxication, or to speak more generally, accident-proneness following upon the drinking of alcoholic beverages

Many of the members of this conference are no doubt familiar with alcohol accident statistics as published and know that in recent years approximately 20 to 40 per cent of cases where drivers licenses have been suspended or revoked the charge has been intoxication (34) [1] Although alcohol is directly mentioned in only 7 to 10 per cent of our fatal highway traffic accidents, it appears to be the belief of traffic commissioners (56) and other informed individuals that probably a fourth to a third of our automobile accidents are at least partly chargeable to alcohol use by the drivers Alcohol statistics at present available are necessarily understatements especially for two reasons In the first place on both the legal and the medical sides it has been difficult to arrive at satisfactory definitions of intoxication (2) (33) A moderate impairment of performance due to the effect of alcohol on the nervous system is not always qualitatively different from the impairment which may come with fatigue, the loss of sleep, illness, or at times from a period of prolonged inattention The relative similarity of behavior mismanagement from different causes (some of them quite usual ones) has tended to draw the line defining actual "intoxication" near the extreme pathological side, thus omitting generally from the group all except the most flagrant cases This minimizes the total statistical import of the alcohol factor as far as the police records are concerned, and it is from these that the alcohol traffic figures are drawn at present In the second place, society has been more ready to wink sympathetically at the drinker than to scowl disapprovingly at him

[1] Numbers in parenthesis refer to bibliography at end

and is inclined when he has a motor vehicle accident to bring a charge of "careless driving," thus sparing him from a more obnoxious designation On the scientific side there is a third important factor operating to confuse recorded statistics, namely that impairment from alcohol does not continue at a constant level and is a characteristically transitory psychological state The results in an examination of a traffic violator depend on his condition at the time of the examination, they cannot show except by inference what the condition was an hour earlier There is now considerable evidence to support the expectation that the use of chemical tests as a means of examining for "intoxication" will partly overcome the difficulties mentioned But the total situation is undoubtedly very complex on the side of personal judgment because so many drivers are their own employers without a sense of employer's responsibility American railroads have adopted a stringent rule "G" forbidding employees to drink or to frequent places where intoxicating liquors are sold But traffic violators do not come under the control of one or of a group of responsible employers, nor do they become subject to traffic commissions until after the misdemeanor has taken place and so preventive measures are not systematically possible

Alcohol's immediate impairment of driving ability is quite generally recognized The amassing of scientific evidence on the effects of alcohol and the fitting together of this evidence with the result of finding alcohol to be a depressant and not a stimulant for mental functions (12) (44) (58) has been reviewed repeatedly (e g , 16) and need not be restated here in detail It will be sufficient for us to remind ourselves of some of the characteristics of behavior, following the ingestion of alcohol, that are pertinent to the task of the motor driver One important effect which is predominantly physiological should be mentioned first Experiments on different kinds of animals agree in showing that muscular incoordination resulting from alcohol appears more strongly in malcontrol of the lower or hind limbs than in the fore limbs or arms Rats and dogs and other animals suitably dosed with alcohol can continue locomotion, of a rather poor type, by means of their forelimbs while actually dragging their hind limbs The human being is, of course, out of the running under similar dosage conditions, and hence it is that the staggering gait of the partially inebriated man has such classical diagnostic significance Because of this marked effect on the legs the automobile is particularly useful to the more or less intoxicated individual He can sit and drive when it would be difficult for him to proceed as a pedestrian The vehicle, guided and driven largely by use of the less affected arms, permits him to make progress that would if he were dependent solely on the more affected legs, be physiologically inconvenient even impossible for him at the time

The mental changes produced by alcohol are numerous and among them are included the alterations in mood and attitude that account for the wide and enthusiastic use of the substance by pleasure-seeking mankind Because these emotional changes are psychologically at least as important as the sensory and muscular alterations which typically follow the ingestion of moderate amounts of beverage alcohol in being at variance with the theoretical normal conditions for the automobile driver, I include them in the following catalogue of behavior patterns of those who drive after drinking The muscular and sensory responses are more amenable to experimentation than the emotional, and, whatever the mood, speed, precision and undivided attention are essential to efficient driving

(1) Experiment shows that reaction times in the responses of eyes, hands, and feet are slower after alcohol ingestion than under similar conditions but without the beverage Measured in the laboratory this slowing of reaction time is approximately 5 to 10 per cent after the usual experimental dosage This change is in itself large enough to contribute to accident proneness and sometimes to present the chief precipitating cause

(2) Less uniformity in response follows the ingestion of alcohol Motor coordinations of the arms and legs are less true to patterns of efficiency and more variable in amplitude when alcohol is influencing the nervous system Under certain circumstances what is essentially the correct coordination may be performed in such a clumsy indirect manner that the result is the equivalent of having done just the wrong thing

(3) A poor grade of attention to diverse external stimuli follows the taking of alcohol The less erect body posture together with modifications of vision and hearing in the direction of poorer perceptual facilities all operate to accentuate the attention decrement This effect of alcohol is sometimes spoken of as a narrowing of the attentional field The well-known tendency of the alcoholized individual to repeat himself or to continue talking interminably on the same topic is evidence of the perseveration occurring in this narrowed attentional field and of the similarly limited ability to turn from one type of stimulus or cue to another

(4) On the emotional side the achievement of increased self-assurance is apt to be the first change noticed, it is probably, often unconsciously, an objective in the use of alcohol Everywhere even a slight socially acceptable toxic effect is recognized as bringing this subjectively desirable feeling to the drinker Many are the claims advanced for the gains achieved among friends by a glass or two of "stimulant," but in highway traffic the characteristic alcohol effect beside making the individual satisfied with his poorer muscular control prompts him to the assumption of the right-of-way and makes him unduly willing to venture on a sporting chance [1]

(5) Further disturbance of emotional balance is a fairly common

[1] The person who knows the scientific side of the alcohol effect but who notwithstanding has imbibed alcohol, as a driver, may employ more cautionary measures than usual, e g stopping at the side when traffic seems thick.

result from imbibing alcohol, it tends to increase as the tolerance level of the individual is passed This modification in the psychology of the driver may take diverse forms, some of which are conceivably desirable, while others are fundamentally undesirable Certainly a person who is by nature inhibited, fearful and over-cautious may be without the use of alcohol a danger on the road, but probably the number of people of this kind who can best alter their temperamental difficulties through the use of alcohol as a preliminary to driving is relatively small because in general other less desirable changes will occur to offset the intended gain On the other hand, the individual who knows from past experience that it is unwise for him to drive after having taken alcohol may, if he has indulged, be made thereby to assume the hampering timid and overcareful manner somewhat as does a man who knows that he is driving without a license Individual differences in emotional effect are quite large Alcohol makes some individuals hilarious, others bad tempered and irritable Paradoxically, the important element in common is variability which characterizes emotional mood after alcohol and this means that habitual attitudes are no longer to be depended upon Whatever its characteristics an undercurrent of unusual feeling is apt to be a liability on the highway

Exact laboratory experiment has been made with reference to the effect of alcohol upon efficiency in the objective part of behavior The subjective has also been studied and qualitative results are available A brief statement regarding findings on the effects of alcohol on typists is introduced here because it best illustrates alcohol effect on the speed and accuracy of individuals highly trained in the particular skill involved Following the findings on the influence of alcohol on typing a short review will be given of all the available experimental studies showing the influence of alcohol on automobile drivers

Five skilled typists all of whom were spontaneously moderate users of alcohol served in the experiments (44) The results are here presented as averages for the group The men took their alcohol in 20 per cent beverages and the amount was administered according to body weight Dose A contained from 21–28 gm of absolute alcohol (depending on the size of the man), and Dose B similarly contained from 32–42 gm Control non-alcoholic drinks were used with the same five men on non-alcohol days and a comparison made of the two sets of experiments Within the first two hours after the smaller dose the typing rate decreased only 2 per cent, but there was an increase of 39 per cent in the errors The pulse rate was 6 per cent faster, the finger response 3 per cent, the eye response 7 per cent, the voice response 4 per cent slower In the second two hours after taking the smaller amount of alcohol there was practically no change in the rate of typing but there was still an average of 15 per cent increase in errors made The pulse rate remained high, average 7 per cent increase and finger, eye and voice all required longer times than the normal for response, 2, 5, and 3 per cent respectively Within the first two hours after taking the larger dose of alcohol, which represented an increase of 50 per cent over the smaller one, a decrease appeared in typing rate amounting to 4 per cent with an increase in errors of 72 per cent The pulse rate was 8 per cent higher than normal, the finger, eye and voice quickness were slowed up 4, 9, and 5 per cent respectively. In the second two hours after taking dose B the typing rate had recovered practically to normal but the errors were still 35 per cent more numerous than under non-alcohol conditions The pulse rate had not gone down but was 9 per cent faster than normal, finger, eye and voice speed had regained somewhat, but they still showed 3, 4, and 2 per cent slower speed than at a similar time after similar work on the days when the non-alcoholic solution had been taken These results are typical and characteristically illustrate alcohol experiments in the psychological laboratory

The questioning public is not satisfied to estimate accident proneness in automobile drivers from results on what perhaps seems to the layman so different a task as typing And so a new alcohol experimentation is necessary that directly involves the use of the automobile Difficulties and actual casualty risks to be overcome in providing conditions for such a scientific experiment are, of course, considerable Experiments on chauffeurs involving the performance of tasks under special conditions devised to parallel actual driving requirements are a first step in the desired direction Recent work along this line still mostly unpublished, has been done by Bahnsen and his associates (3) in Denmark, and a paper by Mayerhofer (40) of Germany is of this nature Mayerhofer studied the effects of 40 cc of absolute alcohol in wine and beer on some twenty individuals, and he summarized the effects of alcohol on reaction time in situations psychologically quite similar to those involved in automobile driving He found the following conditions. (*a*) Lengthening of the reaction time, (*b*) marked increase of the errors in coördination reaction, multiple reaction and disjunctive reaction, (*c*) false judgments of the speed of the movements of other people and of oneself, (*d*) greater expenditure of energy in movement, especially in the case of misperformances, (*e*) decreased attention; (*f*) omission or disappearance of critical blocking or protective inhibition, optimistic judgment of situations, (*g*) tremor of the hand; (*h*) ataxia Heise and Halporn (30) are probably the first to report on a test (prepared by Dietze) involving the actual driving of an automobile (on a closed highway), under otherwise similar alcoholic and non-alcoholic conditions "A car was rigged up so that shooting a gun would give the signal to apply the brakes and this in turn would shoot another gun The knowledge of the speed of the car and the distance apart of the bullet marks on the road furnished a means of measuring reaction time. Also a curved lane marked by corrugated packing boxes, whose configuration could be changed without notice, made quick decisions necessary and prevented the subject from anticipating the signal to stop. By spending several hours with each subject we were able to notice changes in the individual, and were also fortunate in being able to elicit the subjective symptoms." It was found with this apparatus that when travelling at a speed of 30 miles per hour the subject, when he had not had alcohol, required 16 feet on the highway from the time of hearing the signal until he began to apply his brakes. After taking alcohol (1 to 5 ounces of whisky) the distance required for putting on the brakes gradually increased from 16 to 22 feet. "The driving test was ended abruptly when the road rose up in waves before the driver." Heise and Halporn describe the results in Dietze's complete series of tests, as follows: "Our subjects all mentioned dizziness and two of them a sense of unreality as the most prominent subjective sensations. The most striking change was that of the intelligence, particularly the ability for self-criticism. All but one passed the routine (physical) examination for sobriety but all suffered a moderate slowing of reaction time, and all made mistakes such as colliding with boxes and shooting the gun on the brake pedal at the wrong time." These important preliminary experiments should stimulate other investigators to make similar tests under practical driving conditions.

The diagnosis of *"pronounced drunkenness"* usually offers no difficulty to the traffic officers. But in the preliminary stages of intoxication or in the clearing-up interval after a single or a short time of indulgence diagnostic certainty is difficult. For such cases the services of a medical expert are generally regarded as necessary. The latter in addition to information derived from answers to their direct questions take into account the individuals' general appearance, attitude, orientation in space and time, memory, pronunciation of difficult words, condition with respect to the conjunctivae of the eye, reaction of the pupils, gait and standing steadiness, accuracy of hand movements, smell of breath and rate of pulse. Furthermore, the medical examiner has carefully to rule out possible cases in which a simulation of symptoms of intoxication occur, but where actually the individual has not been drinking but is instead suffering from accident or disease. Generally the total picture based on the medical and psychological signs enumerated is fairly adequate for a diagnosis. Even though the total picture is quite clearly indicative of behavior modified by alcohol in the direction of accident proneness when it comes to translating this evidence into terms of legal procedures practical difficulties arise. A clever attorney by taking the separate symptoms one by one, may convince a jury that all were natural changes due to any one of a number of causes, and when the picture is thus broken up it loses its total psychological impression and medical diagnostic value and so perhaps the case is dismissed. Fortunately for progress in diagnosis a new and more objective kind of evidence has been introduced This is the so-called chemical test and to a discussion of its methods and results I wish to devote the remainder of my time today

Let me briefly sketch the physiological process involved when alcohol is ingested by a human being or other animal It is now very well known that when ethyl alcohol is taken into the stomach it begins promptly to be absorbed without waiting until such time as it passes into the small intestines Also without having been changed in any way by bodily processes alcohol quickly appears in the blood and begins to be used in the metabolism Dogiel in 1874 was among the first to show that alcohol could be found in both arterial and venous blood within two minutes after having entered the stomach, and Higgins (31), a former colleague of mine, showed that alcohol begins to be burned in appreciable quantity within 5 to 11 minutes after drinking The prompt appearance of unchanged alcohol in the circulation and its ready diffusion throughout the whole organism makes the interesting substance uniquely accessible for observation and study not only in the breath but also in the body fluids, the saliva, the blood and the urine By the year 1900 the work of several physiologists and chemists, including noteworthily two Frenchmen, Gréhant (27) and Nicloux (46) had resulted in the general finding that the amount of alcohol appearing in the blood approximately parallels the size of ingestion The next few years saw the development of various methods for detecting alcohol in the body fluids and in 1913 appeared an important paper by Schweisheimer (52) in which the claim was made that the psychic condition of men who have imbibed depends on the degree of concentration of the alcohol in their blood Schweisheimer said "With animal experiments, such a proof could naturally not be given But in this research, it is absolutely demonstrated that with abstainers, moderate drinkers and habitual drinkers, the condition of intoxication indicated by such signs as dizziness, numbness, tiredness, and in slighter cases by talkativeness and hilarity, is exactly parallel in increase and decrease with the increase and decrease of the alcohol content in the blood Furthermore, it appears as far as the abstainer and the habitual drinker are concerned, that an equal concentration of alcohol in the blood caused in the former more marked symptoms of drunkenness, in the latter lesser symptoms" (transl) The next experiments directly involving the relation of the psychological to the chemical function of alcohol were made, I believe, almost simultaneously in England by Mellanby and in America by myself Mellanby in his experiment at this time used dogs checking his results on a few men, my own studies were entirely with men and involved carrying out rather elaborate physiological and psychological measurements, the results being checked at intervals against the simultaneous findings for alcohol content in the body fluids The results of Mellanby and my own verified the general finding of Schweisheimer that the toxic effect paralleled the height of the alcohol concentration in the blood The main difference, probably quite an important one, between the German findings and ours have to do with the declining portion of the concentration curve We found the parallel in the toxic effect and the concentration curve on the rising part of the curve and at the maximum But after the maximum had been passed and the curve was declining we found the alcohol symptoms clearing up more rapidly than Schweisheimer's statement had led us to expect Mellanby stated "A dog begins to show signs of intoxication when the alcohol in the blood reaches about 354 cubic mm per hundred grams of blood. At this stage it will probably hit its hind toes against the floor in walking. Its movements will be slower and after a period of excitement its interest in external conditions will be less than usual This only applies to the ascending portion of the curve It has already been stated that the symptoms decline after the maximum of the alcohol in the blood has been reached When the alcohol has declined again to 354 cmm the dog will probably appear almost normal and will certainly be less intoxicated than at the corresponding point on the ascent of the curve" In other words intoxication resulting from short periods of drinking is a kind of mental and motor disorganization arising from interferences with normal nervous system action produced by the invasion of the drug substance into the organism When the strength of the pharmaco-dynamic attack decreases the organism experiences a rather sudden turn for the better A psychological factor that complicates attempts at psychochemical correlation is a partial adaptation that the organism makes even when the toxic load is heaviest As soon as the handicap is reduced even slightly a rebound toward adjustment and organization of functions takes place The Schweisheimer concept, that the toxic effect exactly parallels the rising and falling of the concentration curve, has been practically convenient but it does not fully fit the psychological facts The tendency for the subject to clear up psychologically before alcohol has disappeared from his system may account for the contradictory statements made concerning a particular individual and for the great diagnostic difficulty that exists in examining patients 30 minutes or more after an accident has taken place

The value of the objective chemical test is coming to be generally recognized Recently several studies have been published which give comparisons between the results of chemical analyses of fluid samples and independent symptom diagnoses of degree of intoxication Although alcohol appears in all the body fluids, practical considerations limit the types of materials that may be studied. The most accessible sample is the breath, yet it offers certain difficulties as a point of attack In this country Bogen (8) and in Sweden, Liljestrand and Linde (36) have, however, used it rather extensively In their studies the person to be examined blows up a small rubber sack resembling a rubber pillow and having a capacity of two liters and then the air while still warm is passed through indicating chemicals The test is not entirely reliable because gas coming directly from the stomach may intrude into this exhaled air and thus carry into the sample larger alcoholic content than the same amount of normally mouth breathed air would contain The possibility of this type of error in the sample collection probably applies especially to the period nearest to the drinking of the beverage, and the examiner can not be sure just when his patient last took alcohol It is not uncommon immediately following an accident for a person to have alcohol urged upon him, perhaps even forced between his lips if friends are trying to revive him from a state of unconsciousness or confusion And so the breath test is doubly dubious

Saliva is accessible as fluid to be tested for alcoholic content and it may prove best for the purpose (21) (35) Not a great deal has been done by way of experimentation with it up to the present Blood samples and urine samples, particularly the former present more difficulties on the side of the willingness or unwillingness of the individual under examination, but in spite of the initial psychological reluctance of people to cooperate both of these sample methods are coming into use in connection with automobile traffic and police court activity Where they are used public opinion will no doubt gradually bring about their more common and more ready acceptance The blood sample method is already used by the police doctors of Sweden The lobe of the ear or the tip of the finger of the suspect, after proper non-alcoholic cleansing, is pricked with a needle Small specially prepared capillary tubes ready in a mailing receptacle are filled from the drop of blood that is squeezed out; the tubes are closed and the receptacle is mailed to the central testing laboratory where chemical analysis is made by means of a micro-method devised by Prof. Eric Widmark who has greatly advanced this kind of investigation (62) The preliminary Swedish statistics are of great interest but the work is probably not at present advanced to the point where the boundary line of intoxication can be definitely fixed by this method In addition to the very informing studies of Widmark other publications are appearing in Sweden One on the subject of alcohol and accidents has just come out under the authorship of Hindmarsh and Linde (32) Prof Liljestrand[1] the pharmacologist of Stockholm under whose direction the investigation was carried out has reported this work as follows "From April 1st 1932 to March 31st 1933 such cases as were taken into the Maria Hospital in Stockholm for accidents and put into the surgical clinic were investigated by one of the authors personally and then a blood sample was investigated (by Widmark's method) Some cases—in all 17 per cent—could not be investigated, since they came at times when the two authors were unable to see them, but these cases are all considered in

[1] Personal communication dated November 23, 1933

the calculations to be free from alcohol The number of people investigated was 505 (men, women and children) Among them 125 were found to have alcohol in the blood, 115 men and 10 women In the following therefore only the men are considered The total number of men investigated was 283, thus 41 per cent were found to have alcohol in their blood Of the 283 accidents 113 took place in traffic (motor cars, trains and so on), 50 of them (44 per cent) having alcohol in the blood Of those having alcohol the following table gives some information.

	Alcohol less than 1 pro mille[1]	Alcohol more than 1 pro mille
All accidents	42 (26%)	73 (64%)
Traffic accidents	17 (34%)	33 (66%)

The correlation between the clinical diagnosis and the alcohol in the blood gave interesting results At an alcohol concentration of 1 pro mille 63 per cent were diagnosed as influenced by alcohol, at 1 33 pro mille the corresponding percentage was 78% and at 1 66 pro mille 88% These numbers show that clinical signs of alcohol intoxication were found in this material much earlier than in the material investigated by Widmark in his "Die theoretischen Grundlagen und die praktische Verwendbarkeit der gerichtlichmedizinischen Alkoholbestimmung" (Berlin und Wien 1932) Undoubtedly this is due to the fact that the two authors got a great experience and took much time for each patient (In no case was the diagnosis "influenced by alcohol" without a positive finding in the blood) It is interesting, because it demonstrates that there are obvious symptoms rather early, symptoms that are probably not without danger "

Since alcohol is so highly diffusible it was the theoretical expectation that it would show equal concentration values in blood and in urine for comparable periods following ingestion In my own researches at the Carnegie Nutrition Laboratory in 1919 I trusted this assumption at first and used alcohol measures from urine samples for correlation with results in physiological tests after alcohol ingestion A little later in checking up the comparison between comparable urine and blood samples for my subjects I found that the former gave characteristically a higher alcohol concentration value than the latter, in fact almost 50 per cent higher in the period from 40 minutes to 2 hours after ingestion My data published in 1922 (43) were, I believe, the first to show this difference between the alcohol concentration in the two body fluids importantly used by a number of investigators for obtaining measures of the alcohol in the body The difference relationship that I had found was corroborated with larger quantities of alcohol and over a longer period by Southgate and Carter, in 1926, (55), by Bogen in 1927 and

[1] One mgm of absolute alcohol per cubic centimeter of blood which may be stated as 0 1 per cent

for a larger number of subjects but with similar dosages to my own, by Carlson and associates in 1934 (11) The Carlson study presents for 36 subjects the relation between dosage, blood concentration and urine concentration following in general the method I had employed for my study of nine subjects A formula derived by the Carlson group from a combination of their data with those of Southgate and Carter furnishes a useful practical index for predicting the blood alcohol concentration when the urine alcohol concentration has been measured The formula is as follows blood alcohol equals 0 71 times urine alcohol plus 0 01 The equation derived from my data corroborates the Carlson-Southgate-Carter formula for the range in amount of alcohol concentration covered by both It seems justifiable on the basis of the general agreement of investigators to adopt the practical expedient of using urine samples as an index to alcohol blood concentration at least where there is any legal obstruction to taking blood samples For further details and for the methods of determining alcohol content reference may be made to the original papers cited at the end of this article

Undoubtedly it is rather too early confidently to set the "intoxication line" in terms of blood or urine alcohol content As a program of chemical tests (objective examinations) takes shape it will probably exert some influence in the direction of definiteness on the more subjective side of the examination Many writers at present appear to regard anything lower than one milligram of absolute alcohol per cc of blood, corresponding to $\frac{1}{10}$ of 1 per cent, as permissable and as evidence of a non-intoxicated condition. The present writer has held that this is rather too high if we are trying to pick a safety-first value The results of Hindmarsh and Linde mentioned and discussed above in Liljestrand's communication tend to strengthen this rather more conservative definition since one-quarter of their "alcohol-accident" group appeared within the range falling at or below this value ($\frac{1}{10}$ of 1 per cent) and more than half of the cases included who showed more than 0 1 per cent were diagnosed by these investigators as "influenced by alcohol"

The following personal communication from Dr Larsen[1] who has been using the chemical test at Queen's Hospital in Honolulu is of special practical as well as scientific interest

"We have been using the urine alcohol test for a number of years and have a large number of determinations on both clinical as well as outside alcoholic cases Our findings ran fairly parallel with those of Bogen However, we find the chronic alcoholic can have a much higher alcohol test without clinical symptoms than the individual who is not an habitual drinker

In certain urines there are substances non-alcoholic in type, which if they happen to froth over will give a positive reaction that may suggest the presence of two to three milligrams of alcohol when none is present To avoid this oc-

[1] Personal communication dated October 12, 1933

casional false positive we use a trap between the tubes We have never noticed this caution in the literature, but we feel it might be very important

From the automobile standpoint we feel it is important to prove whether a person has been drinking, for we believe firmly it is not the heavy alcoholic who is the greatest danger in a car, but the one who has merely taken a few 'shots' and considers himself cold sober Therefore a 0 5 mgm test will prove that the man's brain was under the influence of alcohol and his judgment not reliable Perhaps this is old history to you, but here where we have had an unusually high rate of fatalities and definite objection on the part of many to accuse the moderate drinker, we have had to stress this side of the argument

Another point we have made is to take into consideration the specific gravity of the urine to show whether large amounts of water had been ingested before the specimen was obtained We recognize, of course, that occasionally we can have a negative test in the face of some symptoms of alcoholism The test, however, has been very useful in shortening the law process Now both lawyers and clients when they know a chemical test can prove whether they have been drinking or not are much more inclined to confess at once This saves a lot of legal bickering

We have not published our results since we felt that as yet we did not have enough new evidence to warrant a new publication "

It is to be hoped that Dr Larsen and all others in this country and abroad will publish their findings as soon as they become available We wish and need to know each other's results, the technical difficulties as well as the successes that have been met, and not least of all the criticisms of method and treatment that stimulate progress

In presenting the present paper it has been my object to introduce the topic of the relation between alcohol and traffic accidents rather than to settle any points or phases with reference to it The subject is a live one Beverage alcohol has legally returned and will probably continue in many states for years to come Benedict (4) has pointed out the fact that in America alcohol is commonly used as an accessory food, that is, between meals more often than is the case in other countries This imbibing on a relatively empty stomach results usually in a stronger toxic effect relative to the amount ingested Furthermore, it is too frequently the youthfully, thoughtless urge for vivid thrilling experience that brings the alcohol and the speed experiences together temporarily Some individuals are sure to try to employ both of the powerful agencies, alcohol and gasoline at the same time An experienced adult under the mild influence of alcohol can usually drive a modern automobile with some success, but no matter what he claims or thinks about it, probably he is, when even mildly intoxicated, more acceptable to society at large in almost any other rôle than that of motor driver To the army of drivers the right to pursue their way unhampered seems inalienable and to preserve this right safely to the majority objective and thorough means of checking infringements of their privilege by the minority need to be more fully developed and then generally employed The chemical test for intoxication in case of accident is one of the most promising of these.

Highway engineers in the United States are quite aware of the fact that during the last 15 years more of our citizens have been killed in automobile accidents than have lost their lives in military action or have died of wounds in all our foreign and domestic wars Our death toll from the automobile at present is approaching 100 lives per day and for injured nearly 2500 Even though at present we cannot state exactly the percentage value of alcohol's contribution to these figures the fact remains that we know this contribution is quite material Alcohol is among the more specific things that can be pointed to as predisposing causes in the accident picture And yet people of all kinds are interested in facts and especially in the relation of facts to their own comfort and safety They do not want accidents and they want accident makers removed or else taught better traffic habits including especially refraining from drinking before or during the operation of motor vehicles They will be sympathetic to new methods of protecting society on the highroads that are both simple and objectively fair.

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DISCUSSION

ON

ALCOHOL AND MOTOR VEHICLE DRIVERS

Mr Walter W Matthews, *Bureau of Highway Patrol and Safety State Revenue Department, Pennsylvania* I would like to call attention to a paper entitled "Medicolegal Aspects of Drunkenness," which was read before the Medical Society of the State of Pennsylvania in 1932 by Dr. H A Heise and Dr Benjamin Halporn of the Municipal Hospital at Uniontown, Pa Dr Heise has been conducting several tests with the assistance of our department directed primarily towards urinalysis as a measure of presence of alcohol in the system and he has reached some very interesting conclusions In Fayette County, Pennsylvania, in 1924 before the chemical test was used, 34 persons accused of drunken driving were found not guilty and 21 were found guilty or pled guilty He says "In 100 recent consecutive cases in which the alcohol in the urine exceeded 0 19 per cent by weight, 87 were found guilty, 6 were pronounced not guilty but paid the costs, and 7 were acquitted "

Incidentally in the conduct of some of these tests we have furnished highway patrolmen to keep certain portions of the highways more or less free of traffic and to assist as observers, and one of the humorous statements in his report is that he acknowledged the services of, and congratulated, those who have offered to drink whiskey for the sake of science I happen to know that what he did was to ask for volunteers Quite a number of drivers volunteered and he kept feeding the whiskey at intervals until he got up to one-third of a pint per person and then the tests stopped abruptly! Perhaps the most interesting conclusion reached is that it is practically impossible to prove definitely alcoholic intoxication from the symptoms alone One of the things that has to be considered is that drivers may evidently be stimulated by some 16 other biological conditions Just as in the observation, the Doctor speaks of, a typist who was unaccustomed to alcohol, drank one ounce of whiskey, and shortly thereafter was able to type more rapidly than before, but made many more mistakes The next day the experiment was repeated and 15 units of insulin were given a half hour before the alcohol was taken The alcohol curves were identical and the subject displayed many of the symptoms of drunkenness and certainly would have been pronounced intoxicated by the average physician The alcohol in the urine, however, did not exceed 0 015 per cent, giving definite evidence of the value of the test in proving that certain symptoms are not due to alcohol

I would like to hear some discussion on the practical side of handling some of these tests from the police point of view One of the difficulties we have in Pennsylvania is in getting doctors to agree to give such tests because of the fact that they will later be tied up for long intervals in court When they get into court they must qualify as experts Their testimony is often broken down by clever attorneys, and you have a condition whereby discrediting the doctor's testimony and that of the witnesses a man may be proven not drunk, and yet it would be perfectly obvious to a high school girl that her escort in a similar condition was pretty "tight" I would like some of the other gentlemen to explain how some of these intricate tests could be put into actual practice in police work, as it seems to be more and more the assumption that we must have a physician's test to prove intoxication

Then we have on the other hand the legal technicalities involved I have in mind a controversy beyween Pennsylvania and New York State We convict a New York driver in Pennsylvania for, as our law says, "driving while under the influence of intoxicating liquor" We suspend his further driving privilege in Pennsylvania We certify that case to New York on the assumption that New York State will take away the New York license Their law reads that he must be convicted of "driving while intoxicated" and their courts have ruled there is a distinction between driving "while under the influence of intoxicating liquor" and driving "while intoxicated"

There we are again, the technicalities involved in attempted scientific tests, combined with loop-holes in the laws present a very interesting problem from the point of view of actual enforcement

PROF R A MOYER, *Iowa State College* What is the extent of the influence of liquor on the ability of the driver? The test I have in mind is a very realistic laboratory drivers test which Dr Lauer of the Psychology Department at Iowa State College has developed and used during the past ten years He has tested a great many drivers, including the Chrysler test course drivers who were demonstrating Chrysler cars at the World's Fair this summer Although the Chrysler drivers were given a high rating, they were not able to get a perfect rating, which indicates the useful range and reliability of his tests

It seems to me that drivers should be tested in normal condition and then, by varying the amounts of alcohol, should be tested at various stages of intoxication By that means some index would be obtained as to the amount of liquor necessary to make the response of the driver such that he would be considered a dangerous driver I am of the opinion that Dr Lauer would be agreeable to carrying out such tests. He has an excellent set up and I am sure that all of us agree that it is a very important question concerning which we should have as much subjective test data as it is reasonably possible to obtain

Dr Miles: I think Professor Moyer's question is a natural one However we know already the effects of alcohol and unless the opportunity is presented to get a prior rating on individuals who are likely to be intoxicated we cannot afford to waste too much time on just repeats although Dr Lauer and I have talked over the problem of setting up and making some tests with his particular outfit A good many investigators have tested different variations in the dosage and have shown the increasing effect with the increasing dosage

There are individual differences of course Habitual users tend to show somewhat higher amounts of alcohol in the blood after given dosages, in some way or other the adaptation of the body is toward an accelerated appearance of alcohol in the blood rather than the opposite In terms of effect on ten people, if the average is 10 per cent you will find about three that will have an effect of 14 to 17 per cent poorer performance, about three that will have from 9 to 12 per cent, three from 4 to 6 per cent, and one fellow who will have practically zero effect, or perhaps score better than normal But too often we have been willing to stress the extreme individual differences and thus give alcohol the benefit of the doubt Of course we know some men who drink and drive and get home all right, by the law of probability there should be some such cases

Some one has said that the superego is that part of the human that is soluble in alcohol Just the rate of the solubility, etc is a big question but there is no doubt about the totality of the effect We know the gravity of the situation and now we are being faced by insurance companies asking for data on which to rewrite the structure of accident rates because they expect a distinct change

Chairman Marsh I have heard within the last month or so of a proposal to make a test by the amount of alcohol in the air from the lungs Has that ever been investigated thoroughly?

Dr Miles: Yes, it was investigated by Dr Liljestrand in Sweden Such a test was devised by Dr Bogan in Cincinnati It is something that no one can particularly object to The person tested blows up a rubber bag that has a capacity of four or five quarts Then this air (contents of the bag) is passed through some chemicals The trouble is there are other volatile substances given off in the breath that may register on these same chemicals The breath test gives some indication but quantitatively is not dependable,—the urine test is more dependable and the blood test is most dependable

THE PHOTOGRAPHIC METHOD OF STUDYING TRAFFIC BEHAVIOR[1]

By Bruce D. Greenshields
Professor of Engineering Science, Denison University

SYNOPSIS

This report presents a new method of securing accurate data on traffic behavior by means of pictures and illustrates the use of such data in the development of a new formula for expressing the relation between the number of vehicles passing a given point, their average speed, and spacing

From the position of a vehicle in two or more pictures taken at short definite intervals of time its velocity and acceleration can be determined A sixteen minute Simplex moving picture camera was geared for a constant time interval between exposures

From 6,000 pictures of 794 vehicles the following expression for the spacing of cars whose speed is controlled by that of the leading car of a group was developed

$$S = 21 + 1\ 1\ V$$

wherein S is the distance center to center in feet between the vehicles and V is the velocity in miles per hour

This paper presents a new method of securing the accurate data on traffic behavior which are necessary both for the design of streets and highways with adequate capacity and for the proper regulation of traffic The method which involves the use of pictures taken at short, definite intervals of time on motion picture film was developed as the result of an investigation carried out during the past year as a University of Michigan Fellowship project This Fellowship was granted by the Detroit Edison Company

In the first part of the work, the writer had the cooperation of the Michigan State Highway Department which lent the assistance of Frank Olmstead, Assistant Research Engineer, who made valuable suggestions in connection with the development of equipment and the analysis of data During the latter part of the study, the Dow Chemical Company of Midland, Michigan, sponsored and financed an investigation of the effect of road surfaces on vehicle speeds

R L Morrison, Professor of Highway Engineering and Highway Transport, who directed the project has made many valuable suggestions as well as given indispensible and constructive criticism

R S Swinton, Assistant Professor of Engineering Mechanics, has followed the work very closely throughout, given unsparingly of his time and has been especially helpful in developing the necessary technic

The results of the study indicate that the average minimum spacing,

[1] This paper is a portion of a dissertation presented for the degree of Doctor of Philosophy in the University of Michigan

center to center in feet, at which automotive vehicles travel, may be expressed by the formula, $S = 21 + 1.1V$, where S equals the spacing in feet, and V equals the velocity in miles per hour If V is expressed in feet per second, $1.1V$ must be changed to $0.75V$ The coefficient, 0.75, is the brake reaction-time in seconds Brake reaction-time is the time it takes an automobile driver to bring the brakes into operation after he has received a stimulus to do so The data seem to indicate that the reaction-time may vary with driving conditions so as to give different spacings for city traffic and for the open highway The photographic method of investigation may also be used to study practice in passing, the variation in the speed and number of vehicles on each lane of a multiple highway, the time lost by traffic interruptions, and the amount of traffic congestion on a highway

The new method is described and its use is illustrated in the development of a new formula for expressing the relation between the number of vehicles passing a given point, their average rate of speed, and the average spacing at which they travel A brief review of literature pertinent to the subject of speed and spacing is given before the development of the formula After the development and discussion of the formula other traffic problems and the adaptation of the picture method to their solution are briefly outlined.

METHOD OF SECURING AND ANALYZING DATA

By taking pictures of vehicular traffic at short, definite intervals of time an instantaneous record of the position of the vehicles at the end of each interval can be obtained. From the position of a vehicle in each of two successive pictures its velocity may be found, and its acceleration from three or more The spacing of cars in the same picture is evident. In case the following vehicle does not appear in the same picture, but in one at a later interval, the spacing may be fairly accurately determined from its velocity and the interval of time elapsing between the pictures in which the vehicles appear

Field Method

The field method of securing data was quite simple. A 16 mm Simplex movie camera was used to take the pictures An electric motor driven by an automobile storage battery operated the camera with a constant time interval between exposures Figure 1 shows the camera with the motor attachment Varying the voltage by changing the battery terminals controlled the time interval, which might be varied from one-half to two seconds This method was found better than rheostat control The time interval was carefully measured with a stop watch over a period of 40 to 100 exposures and checked by the sweep hand of a photographic timer included in the pictures In order that moving cars might appear in at least two consecutive pictures a field of twice the space traveled per time interval was required. To avoid photographic blur due to motion, a moving car had to be at least 300 feet from the camera. In this case the length of road included in a picture was about 125 feet. The blur might have been lessened by using a faster shutter. At the beginning of each film, and hourly during a run, there was included a photograph of a bulletin board

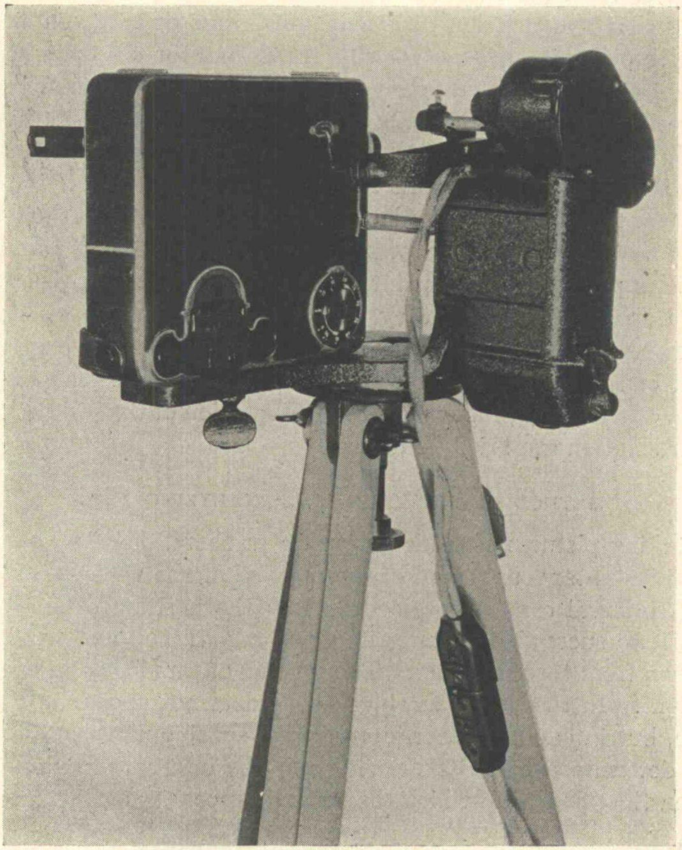

Figure 1. Camera with Motor Attachment

giving the location, date, hour, time interval, shutter opening and other pertinent information.

The white cloth stretched along the opposite side of the road in Figure 2 was used to keep the vehicles from fading into the dark background. Figure 3 shows three frames of pictures taken with the movie camera at this station. The vertical lines are added to show how the pictures look when projected upon a screen with lines drawn upon it for scaling distances.

The measured distance from the camera to the road together with

Figure 2

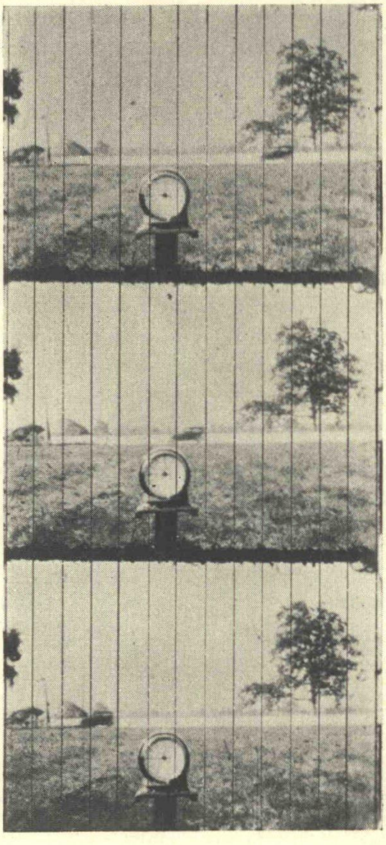

Figure 3

the camera characteristics suffices to give the scale of dimensions which are more accurately determined if the camera is set at right angles to the road As a check, however, a complete plan of the section of the roadway studied is recorded giving the distances from the camera and between objects in the pictures such as fence posts or poles Where no identification exists a 100 foot tape is laid along the pavement and at every 10 foot interval a marker is held over the point and photographed There is thus obtained a definite scale for the picture

Method of Analysis

The process of analysis consists of two parts, first, the pictures are projected upon a white screen ruled with parallel markings and the throw of the lantern is adjusted so that these divisions will represent five or ten foot intervals along the roadway in the picture, and second, the distance of travel or speed and the spacing between cars is taken from two pictures, projected upon the screen at the same time In order to obtain a traffic flow curve for the highway the time may be read from the timer photographed.

The method of studying the pictures may be illustrated from Figure 4 In the first picture at the top the rear car is at point 9 of the scale and 55 feet behind the car in front, measured from front to front of car. In the second picture 0 80 of a second later, it has advanced to point 50 on the scale or has traveled 41 feet. Reduced to miles per hour this is 34 9. By reading the scale in the third picture it is found that the speed of the car is practically uniform, that is, there is no acceleration. To facilitate reading in case accelerations are not wanted, the projection lantern may be mounted on a thin board secured in place by a center pin so that the board and projector may be easily turned to the right or left to bring the car in the first picture to zero on the scale and thus the distance the car has moved, or speed, may be recorded directly in the place of the station

RELATION OF THE SPEED AND SPACING OF VEHICLES

Data secured by the picture method make possible also the derivation of a formula which expresses in a useful way the relation between the number of vehicles in a lane passing a given point, their velocity and the average minimum spacing at which they travel Other valuable studies have been made to determine this relation but it is believed that they have not been based upon as accurate data as it is possible to secure by the picture method It may be reasoned, logically, perhaps, that the driver of an automobile maintains a sufficient spacing between his own car and the one ahead to be able to stop should the first car meet with disaster, but whether he does or not can be found only by observations such as are possible with the photographic method

A review of the literature on the subject of speed and spacing is included for the purpose of comparison and for aid in the interpretation of new data

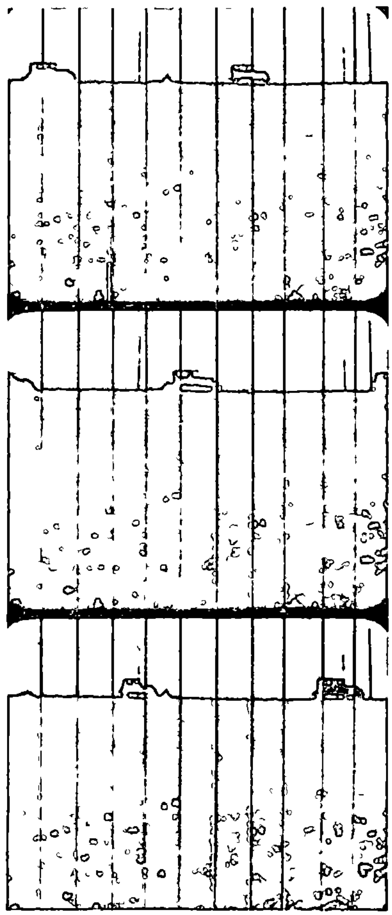

Figure 4

A N. Johnson's Investigations

A N. Johnson, Dean of Engineering at the University of Maryland, derived a formula based upon the idea that cars should maintain a sufficient distance from the car in front for bringing the car to a stop The formula is as follows:

$$N = \frac{5280\,V}{15 + \frac{V^2}{15}}$$

In this formula N is the number of vehicles per hour passing a given point at a velocity of V miles per hour with an average car length of 15 feet, and $\frac{V^2}{15}$ is the clearance between the cars This formula was widely accepted Dean Johnson afterwards, however, (1928)[1] from data obtained from airplane pictures, changed the clearance to vary as the 4/3 power of the velocity This gave the maximum discharge at 34 5 miles per hour as compared to about 15 miles in the first instance This is shown by the curves in Figure 7 The velocities were obtained from the velocities of cars traveling with the traffic and, from the fact that, since adjacent photographs overlapped, the same group of pictures appeared in two succeeding pictures, displaced by the distance that they had traveled between exposures The author states that as the scale of the different photographs varied somewhat, as well as the interval between exposures, the results would be expected to show a like variation in accuracy One hundred and twenty-seven pictures were taken

Sigvald Johannesson's Investigations

Sigvald Johannesson[2] derived a formula based upon "certain observations" of the "Relation of Rate of Speed to Spacing of Vehicles" as follows

Rate of speed miles per hour	Spacing center to center of vehicles in feet
3 0	33
5 0	42
9 5	45
12 0	50

From this he deduced that the "spacing is guided by a certain fixed length, say five feet, plus a time interval, which judged by the recorded vehicle spacings noted above, may be taken to be 1 5 seconds " In other words, the minimum open space between two motor vehicles traveling at a certain rate of speed may be taken to be five feet plus the distance the vehicles will travel in 1 5 seconds at that rate of speed. The equation he then uses for the maximum density of traffic is as follows

$$N = \frac{5280L}{2\,2\,V + 25}$$

[1] Maryland Aerial Survey of Highway Traffic Between Baltimore and Washington, By A N Johnson, Proceedings of Highway Research Board, Vol 8, page 106

[2] Highway Economics, by Sigvald Johannesson Pub 1931 McGraw-Hill page 88

in which N equals maximum density in number of vehicles per hour and V is the velocity in miles per hour According to this equation, the density increases with the rate of speed and converges toward 40 vehicles per minute as a maximum This is shown in Figure 7

N W. Dougherty's Investigations

Professor N W Dougherty, of the University of Tennessee, has evolved an equation[3] for road capacity under the assumptions that all vehicles move in a lane at uniform speed and that they travel at a distance apart to prevent collision if a vehicle in front meets with disaster Three factors are taken into consideration:

(a) overall length of vehicle and clearance, 15 feet,

(b) braking distance, $s = 0\,0259\ V^2$, where s = stopping distance in feet and V = speed in feet per second

(c) time elapsed from the instant in which the driver observes the disaster ahead until he can apply his brakes This time is assumed to be 0 5 second

This gives a reaction distance of $d = 0\,5\ V$ Reducing this to give N, the number of vehicles which can pass a point in one hour at a velocity of V miles per hour, there results the equation·

$$N = \frac{5280V}{15 + 0\,0556\ V^2 + 0\,75\ V}$$

The curve for this equation is shown in Figure 7

The Personal Equation in Automobile Driving[4]

In many fields of activity the presence of the so-called personal equation has been definitely established, and due allowance made for it In few occupations is the existence of the personal equation more evident than in that of automobile driving F A Moss and H H Allen in 1925 carried out an investigation of the personal equation in driving

The object of the tests was to determine

(a) the average time that elapses between the hearing of a signal, such, for example, as the shot of a pistol, and the applying of the brake

(b) the relation between the reaction-time and the variability of the individual

(c) the effect on reaction-time of such factors as the speed of driving, training, age, sex, race, and general intelligence

[3] Roads and Streets, Vol 70, Sept 1930, Page 319

[4] *The Personal Equation in Automobile Driving*, F A Moss and H H Allen, Transactions of the Society of Automotive Engineers, 1925, Part I, Pages 497–510

To carry out these experiments, apparatus was devised that consisted of two revolvers mounted securely on the underside of the running board of an automobile and pointed downward toward the road. One revolver was fired by the experimenter as a signal, and the other by the person under test in making the initial motion of applying the brake-pedal. Shells loaded with red lead were employed, so that when each gun was fired, a bright red spot was made upon the road. The reaction-time was determined from the speed of the car and the distance between the red spots on the pavement. The speed of the car was accurately measured by a chronometric tachometer.

The average reaction-time for the 57 drivers tested was 0.54 second. The variation was from 0.31 second to 1.02 seconds for different drivers. The authors state, however, that owing to the high intelligence of the subjects tested, probably many drivers could be found who might have reaction-times as long as 1.5 or even 2 seconds.

The experiment showed that:

(1) The reaction-time is not appreciably changed with different speeds.
(2) The reaction-time varies little with age and sex.
(3) Persons of a high intelligence seem to have a shorter reaction-time.
(4) Persons who have a shorter reaction-time show the least variation in different tests.
(5) The reaction-time may be reduced by training, the reduction, however, in some men must stop far short of that in others.

Selection of Data

An attempt was made in the investigation of speed and spacing of vehicles to take only such observations as would show vehicles with a speed controlled by the vehicle in front. It is quite evident on a heavily traveled highway where the tendency of traffic is to bunch up, that the speed of the vehicles in the group is controlled by the leading vehicle. When the vehicles are traveling at a higher speed and at greater distance apart it is harder to determine whether the speeds are affected by congestion. It was decided to take pictures only of groups of vehicles which seemed to be driving at controlled speeds and to throw out all observations where the relative velocity of the leading vehicle differed by more than five feet per second with the one following.

Following the method illustrated in Figure 4, about 6000 pictures of both urban and rural traffic comprising 794 vehicles were studied to secure the information used in the derivation of the formula. The observations of spacing for each two mile variation in velocity, shown in the chart on page 391 when averaged and plotted in Figure 5, seem to be fairly well represented by the straight line equation

$$S = 21 + 1.1\,V$$

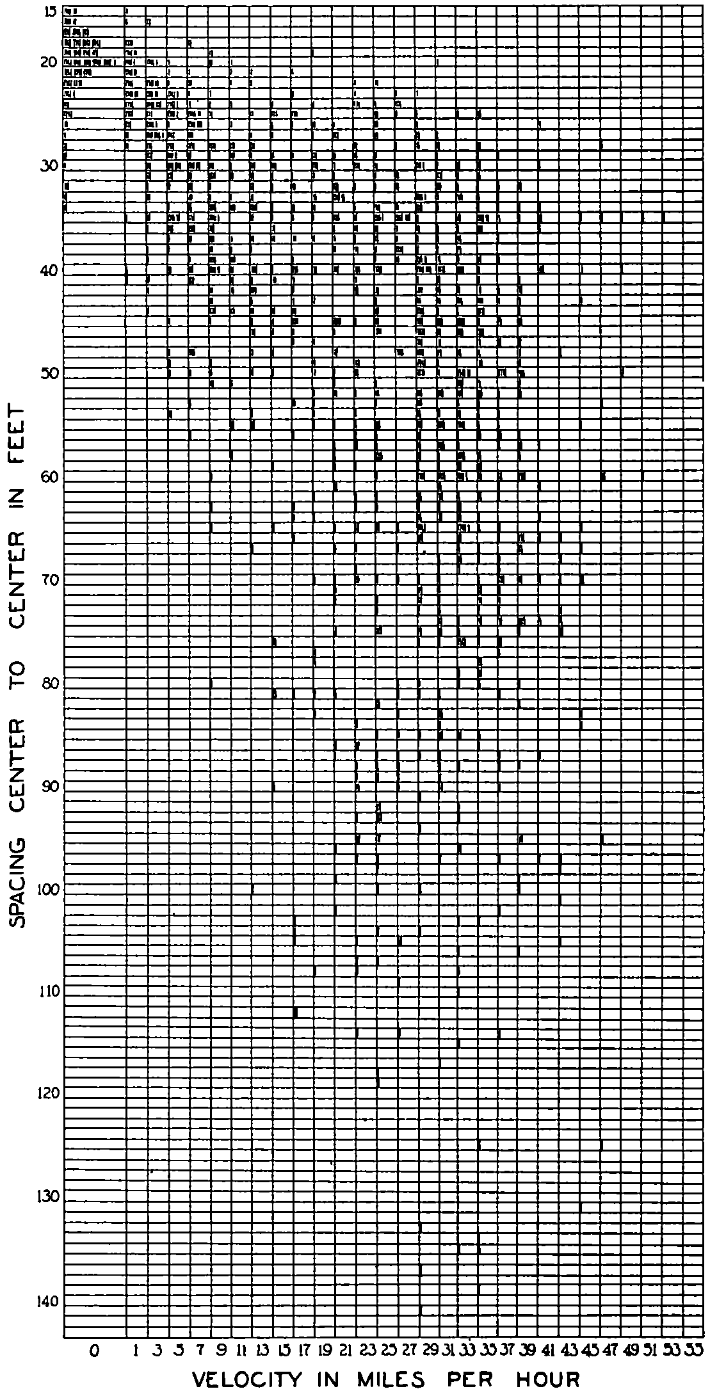

CHART SHOWING VELOCITY, SPACING, AND NUMBER OF OBSERVATIONS where S equals the spacing of vehicles, center to center, in feet, and V equals the average velocity in miles per hour.

Rationalizing the Equation

It was felt that the equation should contain terms for:

(1) the spacing of cars with a velocity at or approaching zero,

(2) an allowance for the distance traveled during the reaction-time of the driver, and,

(3) possibly a term which would take account of the caution or judgment of the driver.

Since the spacings of groups of cars waiting for stop-lights to change had been observed to be from 20 to 22 feet, center to center, giving

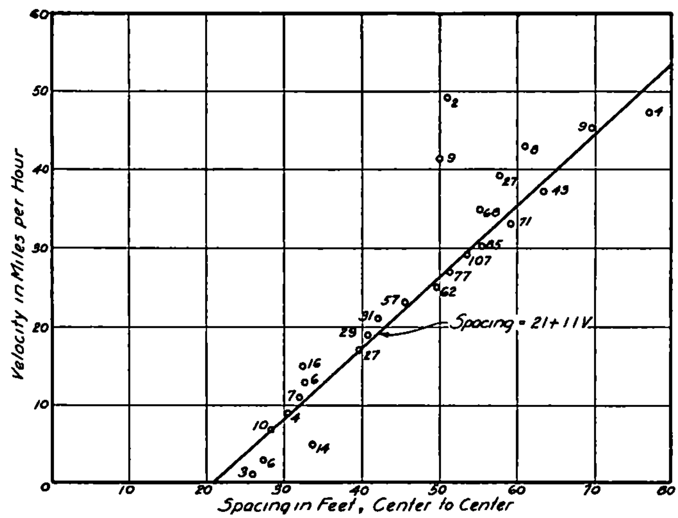

Figure 5. Speed and Spacing of Vehicles. The numbers show the observations for each point.

about a five foot clearance, the first term of the equation, 21, was assumed to be the spacing in feet at zero velocity.

The average reaction-time of 0.54 seconds, found by Moss and Allen, to a stimulus of sound was multiplied by 4/3 to give 0.72 second as the approximate brake reaction-time to a sight stimulus.[5] If the velocity, V, in miles per hour is changed to feet per second the coefficient, 1.1, becomes 0.75, which agrees closely with 0.72, the reaction-time in seconds.

The third term is apparently non-existent. The data, however, comprised in the 794 observations are admittedly lacking both for high and for low speeds. Additional observations of about 500 vehicles, to give a total of 1341 observations are shown in Figure 6. It is noted

[5] Willard Lee Valentine, *A Psychology Laboratory Manuel*, page 41.

that the data for slow speeds from zero to about 15 miles per hour taken from city traffic, seem to determine a line whose equation is

$$S = 21 + 1\,40\ V$$

where V is expressed in miles per hour

This means that reaction-time may be modified by driving conditions or else that the driver, using his judgment, allows himself longer time to react The amount of this variation in reaction-time can be shown only by further study based upon new data

The equation means that the variation in the minimum spacing between vehicles depends entirely upon the reaction-time of the driver Since the negative acceleration of an automobile after the brakes have been applied is practically constant, a car will not collide with the one

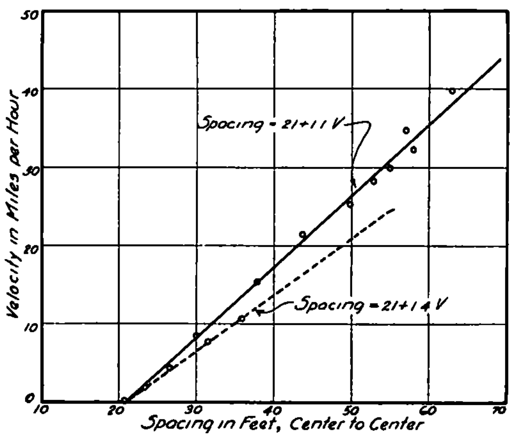

Figure 6 Speed and Spacing of Vehicles The point of zero velocity is the average of 143 observations Each of the other points is the weighted average of 99 observations.

ahead if the brakes are being applied on both cars and if both cars were traveling at the same speed before the brakes were applied A graphic comparison of this formula with those of several other investigators is shown in Figure 7 The number of cars passing over a highway increases with higher velocities so that the theoretical capacity of a highway is increased by one-third if the average speed of all vehicles passing over it is raised from 20 to 40 miles per hour This is an argument for a minimum speed regulation on congested highways

Additional Data on Reaction-Time

It was decided to run a check test of brake reaction-time using a sight stimulus Two cars were parked, one about fifty feet behind the other A chronoscope reading to thousands of a second was wired in circuit with the tail light of the front car and the brake pedal of the rear car, so that it would measure the time from the coming on of the tail light and the pressing of the brake pedal

Each person tested for reaction-time was told to remove quickly his foot from the accelerator pedal when he saw the light flash and push down on the brake pedal as if he were stopping the car. Each person made six or more trials

The average time of 13 individuals tested on one car was 0 86 second, and the average of 27 individuals on another car was 0 74 second

These results agree very closely with the 0 75 second arrived at in the speed-spacing formula, and serve to confirm the conclusion that the coefficient of V in the formula is the brake reaction-time

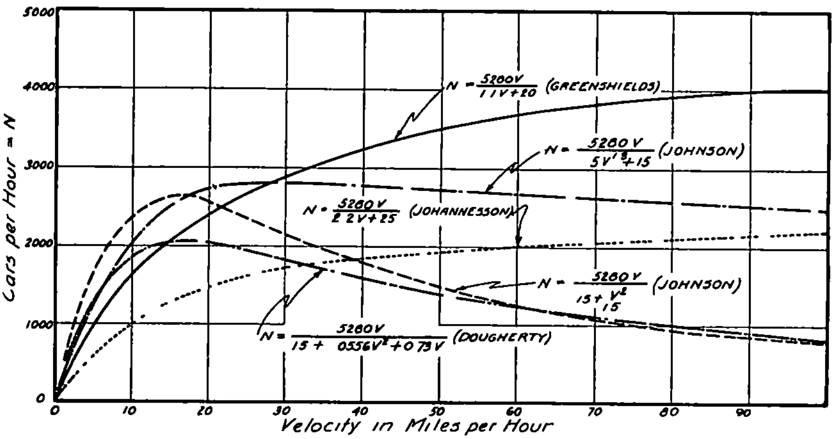

Figure 7. Graphic Comparison of Formulas for Determining the Relation Between the Number of Vehicles Passing a Given Point and Their Rates of Speed.

Significance of the Findings

The spacing between the cars at various speeds is less than has formerly been supposed It is controlled by the reaction-time of the driver This reaction-time may perhaps not be the same at all times for the same individual It is indicated that the reaction-time may be greater in city traffic or else that the driver allows himself more time to react, which amounts to the same thing as far as performance is concerned

Since the two cars tested showed an average difference of 0 2 second in reaction-time for four individuals tested on both cars, the importance of a convenient location of the brake pedal in reference to the accelerator becomes apparent

OTHER PHASES OF TRAFFIC BEHAVIOR FOR STUDY

There are other phases of traffic behavior that could be studied with profit Sufficient work has been done to show that solutions of the problems involved are possible

Passing Practice. At what speeds do cars pass each other? What space and time do they require to pass? Under what conditions do drivers undertake to pass? Pictures taken of a long vista of highway such as could be obtained from a high promontory across a valley should give ideal data on this problem

Amount of Congestion. Congestion starts at the point where the density of traffic is such as to retard its average speed [6] As long as cars can freely pass each other there is no congestion.

The amount of congestion can be found by first determining from freely moving cars, say those over 125 feet apart, the average speed on the highway when it is uncongested. The average speed of vehicles for different densities of traffic will then show the amount of congestion If the average free speed on a highway is 45 miles per hour, a speed of 40 miles per hour for a certain density shows an average loss of five miles per hour per vehicle or a congestion of $\frac{1}{9}$ The composition of traffic should be the same in order to give a true comparison for the average speed of trucks is different from the average speed of passenger cars

Stop-Light Practice What is the total time loss for timed signals and how does this vary with different traffic densities? A study of traffic-actuated signals should be included. Observations of over one thousand cars have been tabulated and a tentative formula, based upon the average acceleration practiced by drivers at stop-lights, has been developed to show the time lost by traffic interruptions

Speeds Attained Under Different Limitations How are speeds related to types of roadway surface, to block lengths, to parking systems, to street widths and to lane of travel? Does lane marking on city streets speed up traffic? In the part of the work sponsored by the Dow Chemical Company, 341 cars passing over a certain section of concrete road from 11 A M to 5:30 P M. had an average speed of 47 8 miles per hour During the same time 64 trucks maintained an average speed of 37 9 miles per hour. On another day 146 cars had an average speed of 47 7 miles per hour. The average speed of 416 cars on a gravel road was observed to be 32 6 miles per hour and of 39 trucks 28 2 miles per hour The weather was dry during the observations and there was no congestion Further studies along this line should be made

Traffic Surveys A device whereby the camera will be operated by a solenoid actuated by a photoelectric cell should be of great aid in getting rather complete traffic information Counting devices operated by the photoelectric cell have already been used successfully The pictures would show the types of vehicles, their speeds and the times of day at which they passed. This information for each car would all appear on one picture since the timer included in the picture would show the time of day while the distance the car had traveled during the

[6] Proceedings of the Highway Research Board, Vol 6, page 218.

small delay from the time it cut the ray of light falling on the cell and the snapping of the camera would show its speed Given the speed of all vehicles and the number passing in any period of time it becomes possible to show the amount of congestion or speed retardation during that period Such a device is now being developed under the direction of Professor Swinton

Not all of the possible uses of the photographic method of investigating traffic behavior have been listed, but its potentialities are evident

This type of study will give pictorial records which should make valuable information available to those concerned in the design of vehicles and the design and maintenance of highways

DISCUSSION
ON
THE PHOTOGRAPHIC METHOD OF STUDYING TRAFFIC BEHAVIOR

PROF J TRUEMAN THOMPSON, *The Johns Hopkins University* I shall briefly describe some work which has been going on under my supervision at The Johns Hopkins University since last June for the United States Bureau of Public Roads

I would not presume to intrude upon this meeting a discussion of my own work were it not for the fact that it bears such a definite relationship to that of Professor Greenshields In addition, since it is too early to advance conclusions or to publish, I feel it should be made a matter of record here to avoid possible duplication of effort

The work to which I refer is also a motion picture study, but in this instance it attempts to discover the lateral placement of vehicles in the act of passing one another, both when moving in opposite directions and when moving in the same direction Obviously, this information is greatly needed To mention only one example of its usefulness, it will probably throw some light upon the moot question as to whether certain classes of vehicles require greater pavement width than do other classes

Another question whose answer we sadly lack is "How much distance does it take for one vehicle to pass another safely when they are traveling in the same direction?" This question derives its importance from the fact that it so definitely influences sight distance and has such a significant bearing upon the limitations which should be imposed upon vehicle length We also tried to answer this question

To accomplish our ends we mounted a motion picture camera outside the driver's window of an automobile in which there were two persons, one to drive and operate the camera, the other to make certain observations and record them This observer's car trailed a vehicle to be studied at about 300 feet and paced it as exactly as possible so that its speed would be known Nothing happened until a second vehicle got in line between the observer's car and the one to be studied, and as soon as it turned out to the left to pass the first vehicle, the film was put in motion and kept going until the passing was completed As soon as the observer's machine got over the point where the second vehicle turned out to pass, a spot of white paint was dropped and a similar spot was made when the observer's machine was over the point where the passing vehicle had turned in again and straightened out The distance between the two paint spots was measured, the speed of the passed vehicle recorded from the speedometer in the observer's car, and the length of the passed vehicle determined by stopping it and measuring it

The transverse placement of the vehicles was obtained from the film by making certain measurements from a still projection of the frame in which the rear axles of the passing and passed vehicles were opposite

DR H C DICKINSON, *Bureau of Standards* Five years ago we analyzed the problem which Dr Greenshields spoke of, about possible spacing of cars on the road There are three factors which he says come into it and as we see it these are (1) the length of the car, for which purpose we used the actual length of the car 15 feet instead of 21, which does not make such difference, (2) the reaction time, which is the time required after the driver knows something is going to happen before he gets his foot on the brake, it averages about 55 hundredths of a second, and (3) the perception time, which depends upon the change in apparent size of the car ahead, because you do not depend upon lights One must see a change in the apparent size of the car ahead or its apparent width on the road in order to know that the car is slowing up. The time necessary to foresee a thing of that kind is considerably longer than is needed to see a light A large number of observations on the road indicate that this time is, or was at that time, approximately one second Thus the spacing on the road should be the length of the car plus the distance travelled in about $1\frac{1}{2}$ seconds, the latter being the sum of perception time and reaction time

Since that time we have been watching the distance between cars on the road It is a simple matter when traveling a distance behind another car, to note the time it takes to reach the point where the car was when you set the stop watch It appears that the *time* between cars is a fairly definite thing It seems to have been shortened materially in the past five years The reason, I believe, is that time between cars is a function not only of the perception and reaction time but also of the safety factor one wants to use When using two wheel and four wheel breaks on the road, one never knew whether the other fellow ahead had four wheel brakes or not, the brake deceleration was not constant and consequently one had to allow a safety factor on that account Now with four wheel brakes almost universal on the road, one drives with that in mind, hence we have discarded that factor of safety. to a certain extent and shortened the time I think Dr Greenshields' figures are close In fact some cars are being driven much closer than that In New York we find cars as close together as 30 feet at 40 miles an hour If traffic is constant and there is no likelihood of an obstruction, no cross roads, there is no reason why one should stop We take chances on that because we think the vehicle won't stop If there are likely to be obstructions or interference, then one must lengthen out the spacing taking into full account the time necessary to put on the brakes The constants in the formula have changed in the last five years

The question of time raises another question which Professor Thompson discussed, that is, how far does it take to pass another car? Obviously, if one car is traveling 1½ seconds behind another car and must accelerate, pass the other car and get back into the road again, it takes a definite amount of time which the engineer can calculate That time calculated for the average car is six seconds, assuming an acceleration rate corresponding to a 10 per cent grade, and that the car starts from a position 1½ seconds behind another and accelerates at this rate until it is back on the right hand side of the road ahead of the other car In order to check this we took two cars and two stop watches—went out on the road and passed each other alternately, the car being overtaken maintaining constant speed While we tried this for speeds from 5 to 45 miles per hour, the time required to perform that maneuver was between 5½ and 6½ seconds—with very few cases outside of that I have checked this figure for five years As a matter of fact it has been my criterion as to personal procedure in driving Coming up behind another car, looking ahead, if I see a car some distance ahead, I decide whether it is safe to pass Of course, the safe distance depends upon speed of the other car as well as my own. I have made many observations of that sort and find that if the time before meeting the approaching car is going to be eight seconds, I will pass If the time is going to be less than eight seconds I will be in doubt—and if I ever pass a car with less than 1½ seconds to spare, I think it is taking chances. This indicates that there is an uncertainty of not more than two seconds in estimating the time of meeting It is a pretty clear-cut geometrical problem and the time required is very closely six seconds The process whereby one makes this fairly accurate estimate, however, involves no material measurement of distance or of time, but is based purely on personal experience and ability to estimate

There is a little more to it than that Instead of starting from your normal distance, 1½ second's time behind another car, you may accelerate and lap the rear bumper of the other car—assuming that only after that point is it impossible to get back if necessary by putting on your brake If one can perform that operation and start a stop watch at the time the cars lap, it takes three seconds from that point until one is clear of traffic in the lane ahead That also is a fairly definite operation.

It can be performed safely by a skillful driver It is a dangerous maneuver, however, since if one sees a car approaching at the instant when his car just laps the one ahead he must decide in a fraction of a second whether to keep on or slow up If he makes a wrong decision it may prove fatal

PROFESSOR GREENSHIELDS The results I gave were averages only In the first pictures taken of traffic leaving a Michigan football game, the spacing was observed to be much closer than that of traffic subsequently observed moving under more normal conditions. The apparent difference of reaction time for rural and urban conditions was pointed out

A study of the average speeds on two-lane pavements of different widths might give some indication of the comfortable width of pavement for drivers of cars and trucks

I might point out that, aside from studies of safety, if the amount of congestion caused by the inability of cars to pass each other is measured that the answer to the question of passing practice is indirectly secured

PROF R S SWINTON, *University of Michigan:* If, for several thousand cars the percentage of cars traveling above given speeds are plotted against speeds on normal probability paper the result will be a straight line If the line is not straight one is comparing unlike things From this line can be anticipated the number of cars that will exceed a particular speed If one finds a range of from 10 to 60 miles per hour he can anticipate the number going over fifty or slower than 20 miles per hour This gives an economic measure of any loss suffered by a few individuals through speed control legislation

THE EFFECT OF CONTROL METHODS ON TRAFFIC FLOW—TRAFFIC MOVEMENTS AT SEVENTEENTH STREET AND CONSTITUTION AVENUE IN WASHINGTON, D C

By E H Holmes

Assistant Highway Economist, U S Bureau of Public Roads

[In Abstract*]

The study was made to compare the time of vehicle passage through an intersection with variable control methods for traffic of nearly constant volume Data were taken between morning and evening rush hours when the traffic was nearly constant at 2,000 vehicles per hour Movement of traffic was studied under no-control, officer control, vehicle-actuated control, and seventeen different fixed-time controls All vehicles that entered and left the zone of influence of the intersection were noted by means of a graphic time recorder From this record the average time per vehicle was determined Over 100,000 vehicles were timed The results are summarized in Table I

From the average time per vehicle tabulated, the following expression was derived to show the result of lengthening the intervals on either or both streets

$$T = 26\,4 + 0\,04x + 0\,28y$$

in which T is the time for the average vehicle to travel 600 feet, including the intersection, x is the green light interval on Constitution Avenue, including three second amber overlap, and y is the green interval on Seventeenth Street, including three second amber overlap

SUMMARY OF CONCLUSIONS

1 For the traffic volume encountered in this analysis (about 2,000 vehicles per hour) operation of the intersection without control incurred the least delay to traffic Of all the control methods, officer control permitted the fastest movement of traffic, closely followed by the shortest fixed-time control, and traffic-actuated control

2 Under fixed-time control, a very marked increase in delay followed lengthening of the cycle In fact, for certain of the short cycles completely reversing the proportioning of the cycle had less effect on the time of the average vehicle than did retaining the same proportioning but doubling the cycle length

3 The flexible control methods showed efficiency equal to or better

* This report has been published in full in Public Roads, Vol 14, No 12 February, 1934

than the most efficient fixed-time control, but it is believed that both the officer control and traffic-actuated control could be more efficient than indicated here

4 During the course of a 10-hour day from 8:00 A M to 6:00 P M. as much delay was incurred during two hours of peak traffic as during the remaining eight hours

TABLE I

AVERAGE TIME PER VEHICLE (SECONDS)

Timing		Constitution Avenue			17th Street			Both Streets
Constitution Avenue	17th Street	West	East	Both directions	North	South	Both directions	
No-control		23 8	25 9	24 9	30 1	28 2	29 1	26 3
Officer control		26 8	29 1	28 0	34 6	33 2	33 9	29 9
15 sec	15 sec	29 3	32 2	30 9	32 6	32 9	32 8	31 5
Traffic actuated		28 9	30 6	29 8	35 3	35 3	35 3	31 6
20 sec	20 sec	31 4	32 4	31 9	33 5	32 0	32 7	32 2
40 sec	20 sec	27 6	28 9	28 3	42 0	41 2	41 6	32 7
30 sec	15 sec	29 5	30 7	30 2	41 1	39 7	40 4	33 5
30 sec	20 sec	30 4	31 3	30 9	41 0	38 3	39 6	33 8
15 sec	30 sec	35 8	38 0	37 0	30 9	29 1	30 0	34 9
30 sec	30 sec	33 7	34 8	34 3	37 2	35 5	36 2	34 9
40 sec	30 sec	31 3	33 3	32 4	41 5	41 7	41 6	35 3
20 sec	30 sec	37 0	36 6	36 8	36 0	34 3	35 1	36 2
60 sec	30 sec	29 9	30 9	30 5	51 4	50 8	51 1	37 1
40 sec	40 sec	36 9	38 0	37 3	38 9	38 9	38 9	37 8
70 sec	30 sec	28 4	29 0	28 7	61 1	55 7	58 3	39 0
50 sec	50 sec	41 7	40 9	41 3	40 6	40 2	40 4	41 0
60 sec	45 sec	35 9	38 4	37 3	51 1	49 5	50 3	41 4
30 sec	50 sec	46 0	46 8	46 4	32 9	32 0	32 5	41 6
60 sec	60 sec	44 1	45 1	44 6	48 6	45 3	47 0	45 4
30 sec	70 sec	56 1	57 3	56 7	32 6	31 7	32 1	48 6
Average traffic (vehicles per hour)								
		625	740	1,365	320	350	670	2,035

In discussion MR J ROWLAND BIBBINS said

I hope further studies will be made under heavy traffic because, from the results shown, it is quite apparent that the rush hour problem is an exceedingly large and difficult one I was impressed with the possibility of the analysis of the individual trip-records by short intervals—15 minutes or even shorter Would not that show what happens in those short intervals with respect to the maximum variations about the day average In that way might be deduced from these trip records some of these essential points of heavy traffic periods

I was gratified to see what a good officer can do when he is properly trained In the fixed-timing tests I noticed that the Constitution Avenue time required was quite low when the proportion of traffic moving in the two streets was passably reflected in the time-split of the cycle Perhaps analysis would show that the officer more nearly apportioned the time-splits between the two movements than the signals

Both stragglers and speeders must be regimented and the movement made more nearly uniform which is, of course, one of the inherent purposes of the progressive signal

One more point Most of us in carrying out investigations like this have no opportunity to use as elaborate mechanism as here available I wonder if a simple notation of "cars delayed,"—what I call a "pile-up"—will not give a rough measure, through a very wide range of operating conditions, of the practical effectiveness of the signal control or any type of control used

I remember on one location in Washington, the Connecticut Avenue Bridge with a 2000 feet clear approach, after 5:25 P M the motor pile-up occurred sometimes one-half or two-thirds way across the bridge, which meant that those end cars would not get through until the third cycle What I want to emphasize is, can we not roughly study traffic control efficiency or adequacy through the determination of pile-ups or percentage of stopped cars?

It will be worthwhile to follow the new Michigan Avenue (Chicago) installation of flexible progressive equipment. The first installation of signals is from 22nd Street north to Oak Street, $3\frac{1}{2}$ miles It will have an expedited rush hour feature, widening the "wave-band" inbound in the morning and outbound in the evening, with automatic reset It is to be hoped that Chicago may be able to experiment with that signal system through a wide range of variable timings and splits as in the Washington tests This paper has shown the necessity of putting such a flexible system through the necessary range of observations

RULES OF THE ROAD IN THEORY AND PRACTICE

By H C Dickinson

Chief, Division of Heat and Power, National Bureau of Standards

SYNOPSIS

In practice there is one underlying principle that is commonly accepted by all good drivers This is that a driver should remain in his own traffic lane and maintain a reasonably uniform speed, only changing direction and speed when he can do so without interference with other traffic

In discussing the application of this principle to the various situations encountered in traffic movements Dr Dickinson points out that in many respects existing traffic regulations are not only at variance with common practices but introduce elements of uncertainty and danger Analysis of the rules in vogue for passing vehicles on four lane roads, for right-of-way at light controlled intersections, boulevard crossings, and for left hand turns, leads to the suggestion that a single regulation based on the foregoing principle, would leave no shadow of a right for two vehicles to be in the same place at the same time; which is not the case with most existing regulations

A careful analysis of the practices of the vast majority of drivers in the various situations which involve the rights and convenience of users of the highway leads to the definite conclusion that one underlying principle is generally accepted by good drivers as a basis for all these practices

Theory as represented by existing traffic regulations based partly on experience in the days of horse-drawn vehicles, and partly on purely academic considerations, is in many respects at variance with common practices and we believe the conflict between theory and practice is responsible for much of the accident toll

The basic principle which appears in all these common practices is that *a driver should remain in his own traffic lane and maintain a reasonably uniform speed, only changing his speed or departing from his traffic lane when he can do so without interference with other traffic in his or other traffic lanes*

Two situations only appear to show complete consistency between regulations and practice as based on the above concept These are (1) meeting and passing other vehicles, and (2) stopping at light controlled intersections In both cases the requirements are definite The driver must remain in his own traffic lane or his own portion of the highway. There is no conflict of rights No two users of the highway may attempt to be *in the same place at the same time.* These two situations are remarkably free from accidents considering the great number of occasions which occur

Applying the principle stated above to other traffic situations entirely different conditions are found

In the overtaking and passing of vehicles on four-lane highways, for instance, arises one of the most serious discrepancies between theory and practice The principle of *lane traffic* would call for each vehicle to remain in one traffic lane except as a change could be made without interference with other traffic Regulations, however, commonly require that a driver must remain in the right-hand lane or must vacate the left lane if another driver wishes to overtake him Despite the regulation drivers recognize the logical practice of overtaking on either side on highways having four or more traffic lanes, and do so freely

The laws should be changed in this respect to correspond with good practice

At intersections not controlled by lights there is necessarily conflict of interests since one driver always must yield to another The application of the logical principle here would require that the driver who must yield right of way should *not interfere at all with the other driver* In place of this simple requirement the most widely used code provides that the driver approaching on the right shall have right of way but that after the driver on the left has "entered the intersection" he shall then have right of way Could a more confusing or dangerous provision be found than this? It encourages beating the other driver to the intersection and transfers the right to cross to the left-hand driver as soon as he enters the intersection

This ambiguity is only somewhat more serious than the general provision which gives the driver on the *right* the right of way at intersections The reason for this instead of the reverse provision has never been clear but only a little study shows conclusively that in this situation the driver on the *left* should have the right of way If this were the case a driver approaching could not enter the intersection until the crossroad on the *left* was clear but having entered he would have right of way to cross because the driver on the *right* of the intersection would be halted

Thus the logical rule for any uncontrolled intersection, not a boulevard crossing, would be as follows *Driver on the left has right of way Driver on the right may not enter the intersection in such manner as to interfere with his right to cross at a normal speed* In case of severe competition, driver entering the intersection must signal for right of way and wait until his signal is recognized and the approaching car slows down to permit his crossing

Where turns are to be made at an intersection there is again a conflict of interests Right-hand turns are not difficult since if made from the right-hand lane they generally do not interfere with other traffic either at controlled or uncontrolled intersections.

As for left-hand turns application of the *lane traffic* principle requires that a left turn be made only when this can be done without interference with other traffic This is the practice of most good drivers despite such regulations as give to the driver making a left-hand turn right of way over approaching vehicles which *have not entered the intersection* Scarcely any driver has the temerity to make use of this provision but occasionally one takes a chance on the other fellow's brakes and may or may not get away with it

For left-hand turns the logical rule is that *the turn shall be made only when this can be done without interfering with other traffic* In case of extreme congestion the turn shall be made only after signaling to the approaching driver for right of way and being assured that the signal is recognized and acted upon

There has been much discussion of the left-turn problem, particularly of the "rotary" turn This latter is perhaps the safest form of turn as it gives complete protection to the turning vehicle but it requires more time than the simpler turn except when there is serious congestion

At boulevard or "stop" intersections the logical rule would require that the crossing should not be made until this can be done without interference with traffic in *either* direction on the main road Confusion exists as to what is meant by the "stop" sign at these crossings In many cases the sign is interpreted to mean simply that the driver must come to a complete stop after which only the usual rules apply, and as noted above this leaves much to be desired This sign should mean that the crossing *must be made only when it can be done without interferring at all with through traffic in either direction* In case of great congestion the usual requirement should hold—crossing to be made only after the driver has signalled for right of way and had *his signal recognized*

When a driver wishes to "weave" from one traffic lane to another, there is no common provision except the unregulated practice to determine how this should be done, but in most cases the obvious demands of safety prevent the driver from neglecting the general rule that he *must not interfere with traffic in the other lane*

These few cases cited seem sufficient to show that one simple logical practice is generally accepted by all good drivers in all situations in traffic This one regulation is *A driver must not depart from his own traffic lane or slow up within it unless he can do so without interferring with other traffic*

In case of congestion where no opportunity can be found to make the necessary maneuver the driver must make a suitable signal and see that this signal is recognized by the other parties involved before making the move

This simple regulation applies with equal force and logic to all the common maneuvers on the highway in conjunction with the proper conventions at crossings It is simple and far more readily understood than most of the various regulations which it would replace, and in particular it has the virtue that in no case is there any ambiguity as to who is in the right Like the safe and simple rule for meeting other cars it leaves no *shadow of a right for two vehicles to be in the same place at the same time*

So long as ambiguous regulations afford any right for two occupants of the highway to attempt to be in the same place at the same time there will be too many collisions and there will be no clear cut means of telling who is responsible for them

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

MINUTES OF BUSINESS MEETING

HIGHWAY RESEARCH BOARD

December 8, 1933

The meeting was called to order at 4:10 P M with Chairman Hamlin presiding

Present

Executive Committee Members

George E Hamlin, Chairman
A T Goldbeck, Vice-Chairman
T R Agg
H C Dickinson
H S Mattimore
H G Shirley
C J Tilden
Charles M. Upham

Director, R W. Crum

Member Organization Representatives

American Association of State Highway Officials
E W James
American Institute of Consulting Engineers
Fred Lavis
American Road Builders' Association
Charles M Upham
American Society of Civil Engineers
H G Shirley
American Society for Testing Materials
Prevost Hubbard
Asphalt Institute
L M Law
Associated General Contractors of America
H J Kirk
Association of Land Grant Colleges
Anson Marston
Eno Foundation for Highway Traffic Regulation
C J Tilden
National Crushed Stone Association
A T Goldbeck

National Paving Brick Association
G F Schlesinger
National Sand and Gravel Association
Stanton Walker
Portland Cement Association
E M Fleming
Society of American Military Engineers
H F Clemmer representing H C Whitehurst
Society of Automotive Engineers
H C Dickinson
Western Society of Engineers
A N Talbot

Minutes of Previous Meeting

Motion That the Minutes of the meeting held December 2, 1932, be approved as printed in the Twelfth Annual Proceedings

Adopted

Interim Actions of the Executive Committee The Director reported that three special projects had been organized with the approval of the Executive Committee These are

(1) A joint committee with the American Association of State Highway Officials on "Roadside Development"
(2) A cooperative project with the Engineering Experiment Station, Iowa State College on "Use of High Elastic Limit Steel as Concrete Reinforcement"
(3) A special project on "Evaluation of Highway Systems"

Motion That the interim actions of the Executive Committee be approved

Adopted.

Annual Meeting Date Consideration was given to the date for holding the Fourteenth Annual Meeting of the Board

Motion That the dates of December 6 and 7, 1934, be designated for the Fourteenth Annual Meeting of the Highway Research Board, subject to the approval of the National Academy of Sciences and subject to unforeseen contingencies

Adopted

Application for Membership.

Motion That the application of the Institute of Traffic Engineers for membership on the Highway Research Board be approved subject to letter ballot of the membership

Adopted

Report of Committee on Nominations The following report of the Nominating Committee was presented by Mr Stanton Walker.

"The Nominating Committee appointed by your Chairman to consider nominations for vacancies on the Executive Committee met on December 7th with the following present Messrs Anson Marston, H C Whitehurst, E M Fleming and Stanton Walker, Chairman

"After careful consideration of the problem before it, the Committee reports the following nominations

Mr A T Goldbeck, Director, Bureau of Engineering, National Crushed Stone Association

Mr Charles M Upham, Engineer-Director, American Road Builders' Association

Mr Burton W Marsh, Director, Safety and Traffic Engineering Department, American Automobile Association

Respectfully submitted,
E M Fleming
Anson Marston
Stanton Walker, *Chairman*"

Motion That the report of the Nominating Committee be approved and that the Director be instructed to cast a unanimous ballot for the nominees

Adopted

Then followed a general discussion of the work of the Board and of ways and means for securing financial support

Motion That a small committee be appointed by the incoming Chairman of the Executive Committee to consider ways and means for securing financial support for the Board

Adopted

Motion That the Director be instructed to send letters of appreciation to those who participated in the program of the Thirteenth Annual Meeting

Adopted

No further business appearing, the meeting adjourned at 5:07 P M

R W Crum,
Director.

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HIGHWAY RESEARCH BOARD