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In this issue: A discussion of new methods for determining traffic capacity of rural highways. (U. S. Route 40, south of Hagerstown, Md.)

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IN THIS ISSUE

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U. S. DEPARTMENT OF COMMERCE

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BUREAU OF PUBLIC ROADS

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New Methods for Determining Capacity of Rural Roads in Mountainous Terrain

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH BUREAU OF PUBLIC ROADS

Reported [1] by O. K. NORMANN, Chief, Traffic Operations Branch, Bureau of Public Roads, JAMES O. GRANUM, Automotive Safety Foundation, and HARRY C. SCHWENDER, West Virginia State Road Commission

The development of traffic capacity information needed for the design of roads in rough or mountainous terrain has required the combined results of a number of independent yet related investigations, including studies of commercial-vehicle weights and performance characteristics, driver passing practices, and the many types of studies normally associated with a capacity analysis such as frequency distributions of speeds and headways under different traffic volume and roadway conditions.

This article is a combined effort of several agencies to develop practical procedures for the application of the results of such studies to a determination of highway needs in mountainous terrain. The procedures are, however, equally applicable to all types of terrain. They provide a significant advance in analytical procedures for determining improvements in traffic-operating conditions through better alinement, reduction in gradient, and the use of truck climbing lanes on the uphill side of steep grades.

IN a comprehensive survey of highway needs, it is essential to establish a certain level of performance for each highway system in a State and then determine the improvements needed to bring the existing systems up to these performance levels. The levels of performance which are established must be feasible and be based on safe and efficient operation of vehicles, with due consideration being given to the future demands of highway transportation.

Performance levels may be measured and also specified in terms of safe operating speeds. Comprehensive studies as well as past experience have shown that drivers demand, and that it is more feasible to construct, facilities that will permit higher operating speeds (1) in level terrain than in rough or mountainous terrain, (2) on primary highways carrying most of the long-distance travel than on local roads where the average trip length is shorter, and (3) on roads of the same system carrying the higher traffic volumes than on those carrying the lower traffic volumes. While the type of service to be provided by a highway under construction is largely an administrative decision, this decision must be based on driver desires and traffic demand.

Once having established the type of service which a highway or a system of highways should provide, it is necessary to specify this service in terms of design speed and operating speed. The *design speed* is a speed determined for design and correlation of the physical features of a highway that influence vehicle operation; it is the safe speed that can be maintained over a specified section of the highway when conditions are so favorable that the design features of the highway govern (*1*).[2] In short, it is the maximum speed that vehicles can safely travel over any section of the highway during extremely low traffic densities. In the design of a highway, the assumed design speed automatically establishes such features as the minimum stopping sight distance, the minimum sight distance at intersections, the maximum curvature, and the superelevation. Other features, such as widths of pavements and shoulders and clearances to walls and rails, are not directly related to design speed but they should be accorded higher standards for the assumed higher design speeds.

The *operating speed* is the highest overall speed exclusive of stops at which a driver can safely travel on a given highway under the prevailing traffic conditions without at any time exceeding the speed which is compatible with the design features of the highway. For this discussion it applies to the conditions during the 30th highest hourly traffic volume for the year under consideration. It is therefore a measure of the type of service which a highway provides during most of the hours of peak flow. The operating speed on an existing highway is affected by the design speed, the traffic volume, and the number of lanes. Also, for two-lane roads, it is affected by the availability of sections on which the sight distance is of sufficient length to permit safe passing maneuvers. In the design of a new highway, it is the one factor which together with the traffic volume and assumed design speed determines the needed geometric features.

Drivers will accept as reasonable a somewhat lower operating speed, or a higher degree of congestion, on a highway that has been in existence for several years than they will accept or expect on a new highway or one recently reconstructed. Also for a needs study to be realistic, there must necessarily be some overlap in the standards by which existing highways are judged for adequacy and those used for a new highway.

In the early stages of the West Virginia Highway Needs Study, the Engineering Committee after reviewing the results of speed studies on highways throughout the State agreed upon a set of tolerable conditions for judging the adequacy of existing highways in order to determine those in need of construction or reconstruction. A set of standards was also prepared for use on new construction. Both were in terms of operating speeds and design speeds. The tolerable conditions and the construction standards for highways in West Virginia carrying over 1,800 vehicles per day are shown in table 1. These conditions and standards were determined prior to the passage of the Federal-Aid Highway Act of 1956.

After these tolerable conditions and standards in terms of service to traffic had been

Table 1.—Tolerable conditions for existing rural highways in West Virginia carrying over 1,800 vehicles per day, and standards for new construction or reconstruction in terms of the service provided [1]

Highway system and type of terrain	Tolerable conditions		Construction standards	
	Operating speed	Design speed	Operating speed	Design speed
Interstate System highways:	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*
Valley or level terrain	45–50	60	50–55	70
Rolling terrain	40–45	50	45–50	60
Mountainous terrain	40–45	45	45–50	60
Other than Interstate System highways:				
Valley or level terrain	45–50	60	50–55	70
Rolling terrain	40–45	50	45–50	60
Mountainous terrain	35–40	40	40–45	60

[1] As determined prior to the passage of the Federal-Aid Highway Act of 1956.

[1] This article was presented at the 36th Annual Meeting of the Highway Research Board, Washington, D. C., January 1957. Mr. O. K. Normann was appointed Deputy Assistant Commissioner, Office of Research, effective September 8, 1957.

[2] Italic numbers in parentheses refer to the list of references on p. 44.

Table 2.—Geometric standards [1] for new construction of two- and four-lane primary rural State highways

Design features	Two-lane highways with future average daily traffic volumes of—												Two-lane highways on Interstate System, and located in—			Four-lane highways located in—		
	Less than 500 ADT, and located in—			500–1,800 ADT, and located in—			1,800–3,000 ADT, and located in—			Over 3,000 ADT, and located in—								
	Valley terrain	Rolling terrain	Mountainous terrain	Valley terrain	Rolling terrain	Mountainous terrain	Valley terrain	Rolling terrain	Mountainous terrain	Valley terrain	Rolling terrain	Mountainous terrain	Valley terrain	Rolling terrain	Mountainous terrain	Valley terrain	Rolling terrain	Mountainous terrain
Design speed.....m. p. h..	50	40	35	60	50	40	70	60	50	70	60	50	70	60	60	70	60	60
Operating speed.....m. p. h..	40–45	35–40	30–35	45–50	40–45	35–40	45–50	45–50	40–45	50–55	45–50	40–45	50–55	45–50	45–50	50–55	45–50	45–50
Maximum curvature.....degrees..	9	14	20	6	9	14	4	6	8	3	5	7	3	5	5	3	5	5
Stopping sight distance.....feet..	350	275	225	525	350	275	700	525	350	700	525	350	700	525	525	700	525	525
Lane width.....do....	10	10	10	[2] 11	[2] 11	[2] 11	12	12	12	12	12	12	12	12	12	[3] 12	[3] 12	[3] 12
Shoulder width:																		
In cut.....do....	5	3	3	6	4	4	8	6	4	8	8	4	8	8	4	8	8	4
On fill.....do....	7	5	5	8	8	7	10	8	8	10	10	8	10	10	8	10	10	8
Percentage of passing sight distance available:																		
1,500 feet.....percent..	(4)	(4)	(4)	10	5	------	10	5	------	10	5	------	10	5	------	(5)	(5)	(5)
1,000 feet.....do....	(4)	(4)	(4)	------	------	5	------	------	[6] 5	------	------	[6] 5	------	------	[6] 5	(5)	(5)	(5)
Grade [7].....do....	5	7	9	4	5	7	7	7	7	7	7	7	7	7	7	7	7	7
Right-of-way width [8].....feet..	60	60	60	80	80	80	100	100	100	120	120	120	120	120	120	200	200	200
Surface type.....	Low	Low	Low	Medium	Medium	Medium	High	High	High	High	High	High	High	High	High	High	High	High
Bridges: [9]																		
Loading [10].....	H15–S12	H15–S12	H15–S12	H15–S12	H15–S12	H15–S12	H15–S12	H15–S12	H15–S12	H20–S16	H20–S16	H20–S16	H20–S16	H20–S16	H20–S16	H20–S16	H20–S16	H20–S16
Minimum vertical clearance.....do....	14.5	14.5	14.5	14.5	14.5	14. 5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5	14.5

[1] As determined prior to the passage of the Federal-Aid Highway Act of 1956.
[2] On new construction or reconstruction, 12-foot lanes should be used with volumes above 1,200 vehicles per day when truck traffic is more than 5 percent.
[3] All 4-lane highways shall be divided. Whenever possible the median shall be at least 20 feet wide, and in no case less than the 4-foot barrier type.
[4] Percentage feasible.
[5] Not applicable.
[6] Five percent or as required for capacity.
[7] For 2-lane highways carrying less than 500 vehicles per day, add 2 percent for grades under 750 feet long; for 2-lane highways carrying from 500 to 1,800 vehicles per day, add 1 percent for grades under 750 feet long; for 2-lane highways carrying 1,800 and more vehicles per day and Interstate System routes, the 7-percent grade is maximum unless percentage and length of grade are determined by capacity calculation.
[8] Control of access shall be provided on all new locations on the Interstate System and on other primary highways carrying large volumes of traffic.
[9] For bridges carrying less than 3,000 vehicles per day, the width of the structure, if less than 50 feet long, shall be the full roadway width; if the length of the structure is greater than 50 feet, the width shall be that of the approach pavement plus 4 feet and the width of the median in the case of 4-lane divided highway.
[10] For highways carrying less than 3,000 vehicles per day, the loading shall be H15–S12 or one H20 truck, whichever produces the greater stress; for highways carrying 3,000 or more vehicles per day and a volume of heavy truck traffic comparable to Interstate System routes, the loading shall be H20–S16.

Table 3.—Tolerable conditions [1] for two- and four-lane primary rural State highways

Design features	Two-lane highways with future average daily traffic volumes of—												Two-lane highways on Interstate System, and located in—			Four-lane highways located in—		
	Less than 500 ADT, and located in—			500–1,800 ADT, and located in—			1,800–3,000 ADT, and located in—			Over 3,000 ADT, and located in—								
	Valley terrain	Rolling terrain	Mountainous terrain	Valley terrain	Rolling terrain	Mountainous terrain	Valley terrain	Rolling terrain	Mountainous terrain	Valley terrain	Rolling terrain	Mountainous terrain	Valley terrain	Rolling terrain	Mountainous terrain	Valley terrain	Rolling terrain	Mountainous terrain
Design speed.....m. p. h..	40	35	30	50	40	35	60	50	40	60	50	40	60	50	45	60	50	45
Operating speed.....m. p. h..	35–40	30–35	25–30	40–45	35–40	30–35	45–50	40–45	35–40	45–50	40–45	35–40	45–50	40–45	40–45	45–50	40–45	40–45
Maximum curvature.....degrees..	14	20	25	9	14	20	6	9	14	5	7	11	5	7	9	5	7	9
Stopping sight distance.....ft..	275	225	200	350	275	225	475	350	275	475	350	275	475	350	300	475	350	300
Lane width.....ft..	9	9	9	10	[2] 10	[2] 10	10	10	10	10	10	10	10	10	10	10	10	10
Shoulder width:																		
In cut.....ft..	3	3	2	6	3	3	6	4	4	6	4	4	6	4	4	6	4	4
On fill.....ft..	3	3	3	6	4	4	8	6	4	8	6	4	8	6	4	8	6	4
Percentage of passing sight distance available:																		
1,500 feet.....pct..	(3)	(3)	(3)	(3)	(3)	(3)	10	5	------	10	5	------	10	5	------	(4)	(4)	(4)
1,000 feet.....pct..	(3)	(3)	(3)	(3)	(3)	(3)	------	------	[5] 5	------	------	[5] 5	------	------	[5] 5	(4)	(4)	(4)
Grade [6].....pct..	7	8	10	5	7	8	8	8	8	7	7	7	7	7	7	7	7	7
Surface type [7].....	Low	Low	Low	Low	Low	Low	Medium	Medium	Medium	Medium	Medium	Medium	Medium	Medium	Medium	Medium	Medium	Medium
Bridges: [8]																		
Loading.....	H15	H15	H15	H15	H15	H15	H15	H15	H15	H15	H15	H15	H15	H15	H15	H15	H15	H15
Minimum vertical clearance.....ft..	13	13	13	13	13	13	13	13	13	13	13	13	13	13	13	13	13	13

[1] As determined prior to the passage of the Federal-Aid Highway Act of 1956.
[2] Lane widths of 9 feet may be accepted as tolerable for volumes under 800 vehicles per day with a small percentage of trucks.
[3] Not required. [4] Not applicable. [5] 5 percent or as required for capacity.
[6] For two-lane highways carrying less than 500 vehicles per day, add 1 percent for grades under 1,000 feet long; for two-lane highways carrying from 500 to 1,800 vehicles per day, add 1 percent for grades under 750 feet long; for two-lane highways carrying 1,800 or more vehicles per day and Interstate System routes, the 7-percent grade is maximum unless percentage and length of grade are determined by capacity calculation.
[7] For highways carrying 500 or more vehicles per day, a good road surface condition is required; control of access is not required.
[8] For bridges carrying less than 500 vehicles per day, the width of the structure should not be less than the width of the approach pavement; for bridges carrying 500 or more vehicles per day, the width of the structure should not be less than the width of the approach pavement plus 2 feet.

'able 4.—**Average daily capacities of two-lane highways with 12-foot traffic lanes and constructed to a given design speed** [1]

Percentage of highway with passing sight distance [2] of—		Valley or flat terrain with operating speed, 50–55 m. p. h.; design speed, 70 m. p. h.	Rolling terrain with operating speed, 45–50 m. p. h.; design speed, 60 m. p. h.	Mountainous terrain	
1,500 feet	1,000 feet			Interstate System with operating speed 45–50 m. p. h.; design speed, 60 m. p. h.	Other than Interstate System with operating speed, 40–45 m. p. h.; design speed, 60 m. p. h.
		V. p. d.	*V. p. d.*	*V. p. d.*	*V. p. d.*
100	100	4, 850	6, 500	5, 500	6, 550
80	90	4, 450	5, 850	5, 000	6, 000
60	80	3, 950	5, 050	4, 300	5, 300
40	70	3, 350	4, 200	3, 600	4, 600
20	60	2, 400	3, 450	2, 900	3, 850
0	50	1, 300	2, 600	2, 200	3, 050

[1] Based on 5-percent truck traffic during 30th highest hour (12 percent of ADT).
[2] Percentage of 1,500-foot passing sight distance is used for all operating speeds except those below 45 miles per hour. The 1,000-foot values are applicable to all operating speeds.

greed upon, it was relatively easy to es- ıblish the design requirements for new con- ruction and tolerable conditions, as shown ı tables 2 and 3, from the information now ›ntained in the AASHO policy on *Geometric esign of Rural Highways* (*1*) and to prepare ıble 4 from the results of traffic operation and ıpacity studies conducted during the past ›veral years.

Table 4 shows the average daily traffic ›lumes that can be accommodated by a vo-lane highway constructed to a given ›sign speed with various percentages of the ighway having sight distances in excess of 500 and 1,000 feet. The values in this table ·e for the following average conditions ›pplicable in West Virginia:

1. The 30th highest hourly volume during ıe year is 12 percent of average daily traffic ›r that year.
2. During the 30th highest hourly volume : a year, trucks with dual tires account for percent of the traffic.
3. In a capacity sense, the average dual-tired truck is equivalent to 2 passenger cars in valley or level terrain, to 4 passenger cars in rolling terrain, and to 8 passenger cars in mountainous terrain.

For highways where these average conditions do not exist or are not expected to be present during the year for which the highway is designed, appropriate corrections must be made in the capacities by the application of factors similar to those included in the discussion on capacities of existing highways.

Design Speeds of Existing Highways

If the AASHO definition of design speed were applied to existing highways with profiles and alinements that were constructed prior to the time that this term came into common usage, it would be found that in many cases the average running speed of traffic would be several miles per hour above the design speed. Likewise, recent studies have shown that highways constructed to modern standards may provide radically different operating conditions even though their traffic volumes and design speeds are identical. Average speeds, for example, will be much higher on a highway with few 5-degree curves and a lot of tangent alinement than on a highway with many 5-degree curves and little tangent alinement. This is because above-minimum design values are utilized where feasible and drivers do vary their speeds to a considerable extent with the immediate geometric conditions rather than adopting one uniform speed for the entire length of a highway.

Conversely, for a given operating speed, a highway with few curves and mostly tangent alinement will accommodate higher volumes of traffic than a similar highway with many curves of the same degree and less tangent alinement. In relating the operating speed of a highway to its capacity, it is therefore necessary to determine the average highway speed, especially for existing highways.

The introduction of the term "average highway speed," which is in effect the average maximum safe speed or the operating speed for a passenger car over a section of highway during extremely low traffic densities, is an approach which has not previously been employed in relating alinement and profile to capacities. It is, however, an approach which must be employed to obtain reasonable accuracy in capacity determinations, especially for existing highways.

The average highway speed of an existing highway may be determined by weighting the possible speeds of traffic on the individual sections during low traffic flows by the length of the sections. The possible speeds for various horizontal curves and stopping sight distance conditions may be determined by use of the AASHO tables (*1*) relating these features to the design speed.

When preparing plans for a highway, the designer should base the geometric features on an assumed design speed over a substantial length of highway to obtain a balanced design. Invariably there are sections where the designer utilizes values that are adequate for higher speeds than the design speed which he has assumed. The lower the design speed, the greater is the likelihood of the occurrence of such sections. As a result, the high-speed driver can travel over the section during low traffic densities at an average speed which exceeds the assumed design speed. This speed is the average highway speed and is equivalent to the low-volume operating speed.

Figures 1 and 2 show how the operating speeds on a two-lane highway vary with the average highway speeds, the percentage of highways having 1,500-foot passing sight distance, and the traffic volumes. The average daily traffic volumes in these charts are based on highways located in essentially level terrain, with 12-foot traffic lanes, carrying 5-percent dual-tired vehicles with a passenger-car equivalent of 2, and a 30th highest hourly volume during the year of 12 percent of the average daily traffic. The charts were prepared for the Tennessee Programing Study from the information contained in table 5

'able 5.—**Average daily capacities of two-lane highways located on level terrain and carrying 5-percent truck traffic during the 30th highest hour** [1]

Operating speed	Percentage of highway with passing sight distance of—		Average highway speed of—					
	1,500 feet	800 to 1,000 feet	. 70 m. p. h.	60 m. p. h.	55 m. p. h.	50 m. p. h.	45 m. p. h.	40 m. p. h.
M. p. h.			*V. p. d.*	*V. p. d.*	*V. p. d.*	*V. p. d.*	*V. p. d.*	*V. p. d.*
50–55	100	100	4, 850	4, 750	4, 300	---------	---------	---------
50–55	80	92	4, 450	4, 150	3, 750	---------	---------	---------
50–55	60	84	3, 950	3, 450	3, 000	---------	---------	---------
50–55	40	76	3, 350	2, 700	2, 250	---------	---------	---------
50–55	20	68	2, 400	1, 800	1, 350	---------	---------	---------
50–55	0	60	1, 300	900	550	---------	---------	---------
45–50	100	100	7, 100	7, 100	6, 600	5, 650	---------	---------
45–50	80	90	6, 800	6, 400	5, 750	5, 050	---------	---------
45–50	60	80	6, 400	5, 550	4, 800	4, 000	---------	---------
45–50	40	70	5, 750	4, 600	3, 900	2, 800	---------	---------
45–50	20	60	4, 900	3, 750	2, 800	2, 000	---------	---------
45–50	0	50	3, 800	2, 800	2, 000	1, 250	---------	---------
40–45	100	100	8, 450	8, 450	8. 050	7, 450	6, 550	---------
40–45	80	87	8, 250	7, 700	7, 300	6, 700	5, 800	---------
40–45	60	76	7, 900	6, 800	6, 450	5, 700	4, 850	---------
40–45	40	64	7, 350	5, 900	5, 300	4, 600	3, 700	---------
40–45	20	52	6, 650	4, 950	4, 100	3, 200	2, 200	---------
40–45	0	40	5, 850	3, 950	2, 700	1, 950	1, 250	---------
35–40	100	100	9, 900	9, 900	9, 900	9, 600	9, 100	7, 900
35–40	80	85	9, 750	9, 350	9, 000	8, 750	8, 200	7, 150
35–40	60	72	9, 400	8, 650	8, 100	7, 950	7, 250	6, 000
35–40	40	58	8, 950	8, 000	7, 150	6, 950	6, 150	4, 700
35–40	20	44	8, 400	7, 350	6, 300	5, 850	4, 500	3, 100
35–40	0	30	7, 650	6, 600	5, 350	3, 850	2, 450	1, 500

[1] A truck factor of 2.0 for West Virginia; the normal factor is 2.5. The 30th highest hourly volume is 12 percent of ADT.

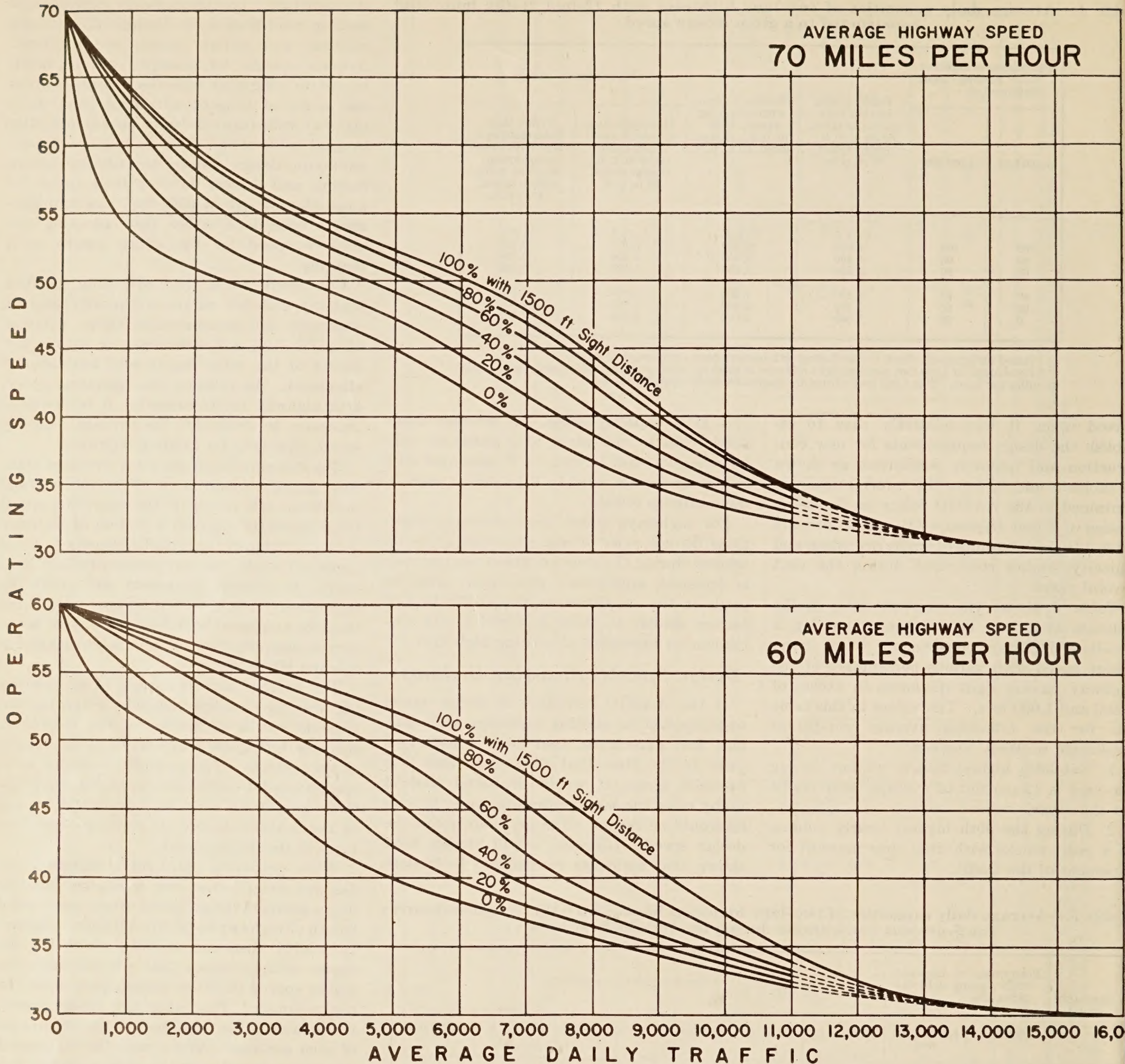


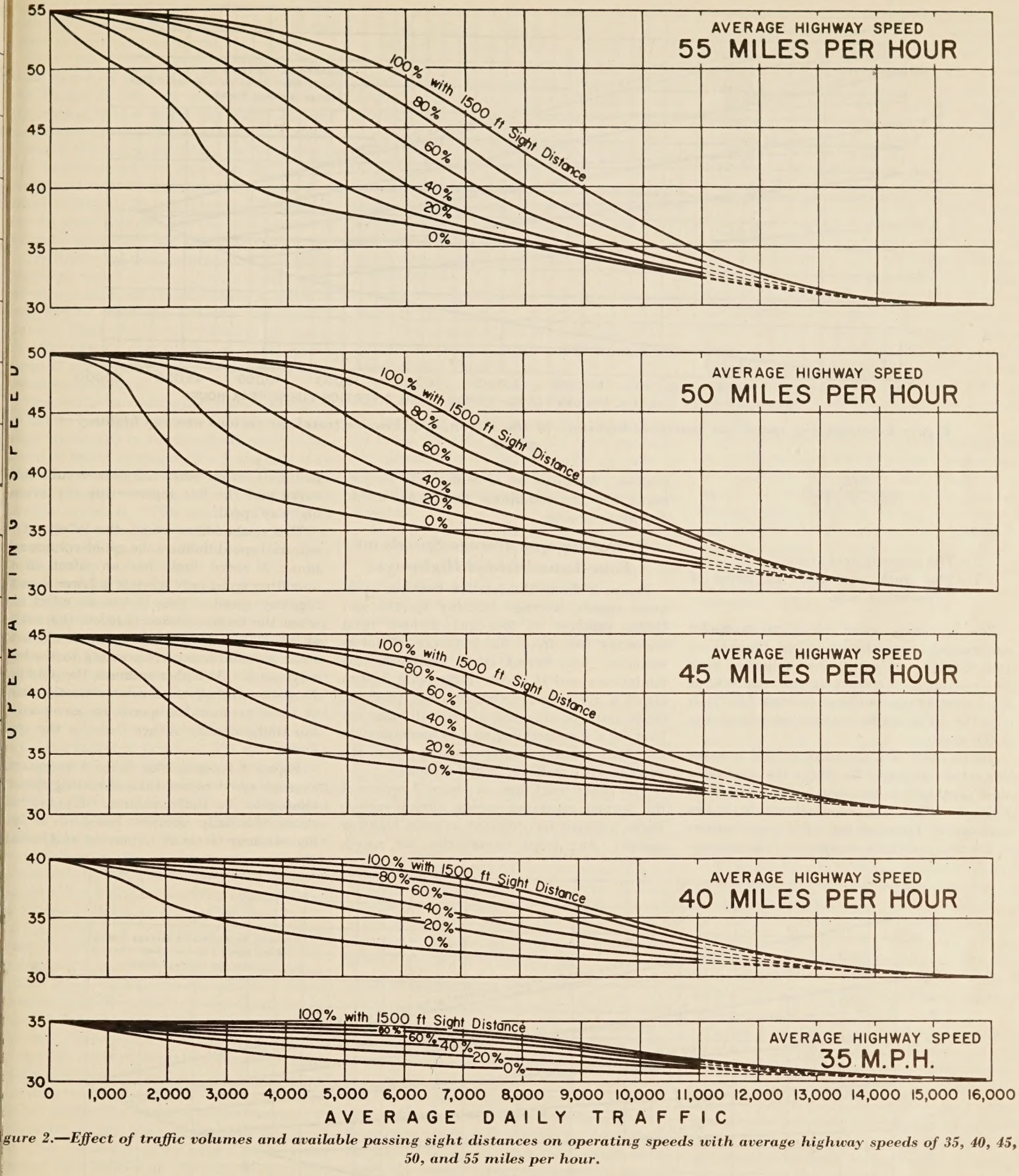
Figure 1.—Effect of traffic volumes and available passing sight distances on operating speeds with average highway speeds of 60 and miles per hour.

which was prepared for the 1953–54 West Virginia Needs Study. Table 5 in turn was prepared from the results of extensive highway capacity studies conducted by the Bureau of Public Roads in cooperation with the various State highway departments and includes the results reported in the *Highway Capacity Manual* (*2*) supplemented by more recent investigations.

In figures 1 and 2 there are curves representing roadways with sight distances that are continuously in excess of 1,500 feet to those which have no sections with 1,500-foot sight distance. The relation between operating speed and traffic volume as shown by the curves is applicable, however, only when the percentage of the highway not having a 1,500-foot sight distance is fairly evenly distributed between the limits of 1,500 feet and the stopping sight distance for the design speed. This is the more usual condition.

It must be pointed out that most of the data on which figures 1 and 2 are based were obtained by studies conducted during traffic volumes within the lower three-quarters of the range (below 12,000 ADT). Studies conducted on two-lane highways during capacity volumes represent principally level tangent sections well removed from sharp horizontal or vertical curves. For this reason, all except the top curves (100 percent with with 1,500-foot sight distance) are shown as broken lines for traffic volumes above 11,0 vehicles per day. There is still considerab question as to whether all the curves for t same average highway speed meet at a co mon point on the right, or whether the possib capacity and the speed at this capacity a slightly lower for the highways with t poorer alinement than for those with a co tinuous sight distance in excess of 1,500 fe This, however, is not too important a co sideration because the practical capacities two-lane highways are well within the ran for which reliable data are available.

The charts may be used either to determi the operating speed for a given traffic volun or the traffic volume which the highway w



gure 2.—Effect of traffic volumes and available passing sight distances on operating speeds with average highway speeds of 35, 40, 45, 50, and 55 miles per hour.

commodate at a given operating speed. hen it is desired to determine the capacity a given operating speed for lane widths her than 12 feet, for 30th highest hourly ctors other than 12 percent, for truck perntages other than 5 percent, or for truck uivalents other than 2, the following procedure must be applied to adjust the capacity volumes to the prevailing or estimated future conditions:

1. For 11-foot lanes multiply the volumes by 0.86, and for 10-foot lanes multiply by 0.77.

2. When the 30th-highest-hour factor is other than 12 percent, multiply the volumes by 12/actual percentage.

3. When there is other than 5 percent trucks during the peak hour or the truck equivalent is greater than 2, as it will be on grades and in rolling or mountainous terrain, multiply the volumes by—

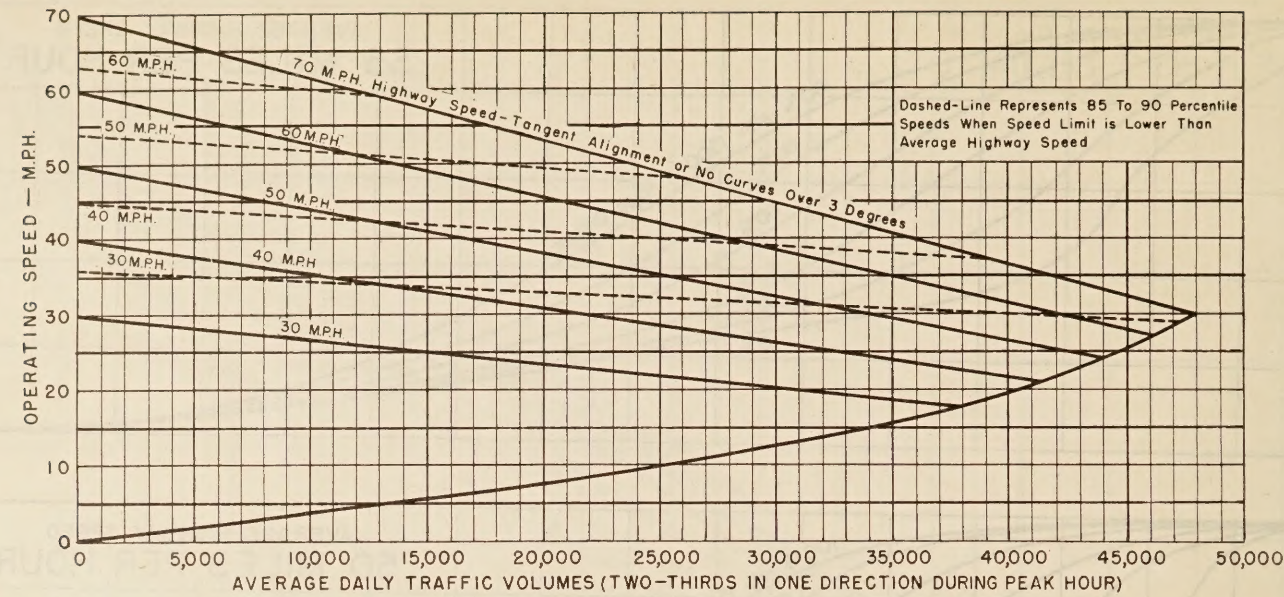


Figure 3.—Operating speeds on four-lane highways in the direction of heavier travel for various average highway speeds.

$$\frac{105}{100-P+PT}$$

Where:

P = The percentage of trucks.
T = The truck equivalent in terms of passenger cars.

The operating speed for a given traffic volume, when conditions other than those used for figures 1 and 2 are applicable, may be determined by employing the reciprocal of the correction factors shown in items 1 through 3 to the given traffic volume before entering in the chart.

Tables A–H are included at the close of this article on pages 38–39 for the conditions most prevalent on two-lane highways in West Virginia. The number of charts or tables that can be prepared for other combinations of the many variable conditions is almost unlimited. A similar set of tables may be prepared for the conditions prevailing within any State or area.

Operating and Average Speeds on Four-Lane Divided Highways

Figure 3 shows the relation between operating speeds, average highway speeds, and traffic volumes on four-lane divided rural highways free from the influence of intersections. The lowest curve represents the minimum speed at which traffic must flow to attain a given traffic volume. For example, traffic must be traveling at least 10 miles per hour for a four-lane highway to accommodate the 30th highest hourly volume when the average daily traffic is 25,000 vehicles.

The other solid lines in figure 3 represent the normal operating speeds during various traffic volumes for different average highway speeds. Any point representing the speed-volume relation must fall between the low curve and the line representing the avera highway speed.

The dashed-lines show the effect of enforced speed limit on the speed-volume re tion. A speed limit has an effect on t operating speed only when it is lower than t highway speed. Also, it has an effect o when the traffic volume is below the volu at which the dashed-line (speed limit) int sects the solid line corresponding to the hi way speed. At higher volumes, the solid li show the normal speed-volume relation, si at these volumes the speeds are governed the traffic density rather than by the sp limits.

Figure 4 is similar to figure 3 except t average speed rather than operating speed related to the traffic volume. Figure 3 a shows the daily volumes based on a 30 highest-hour factor of 12 percent and inclu

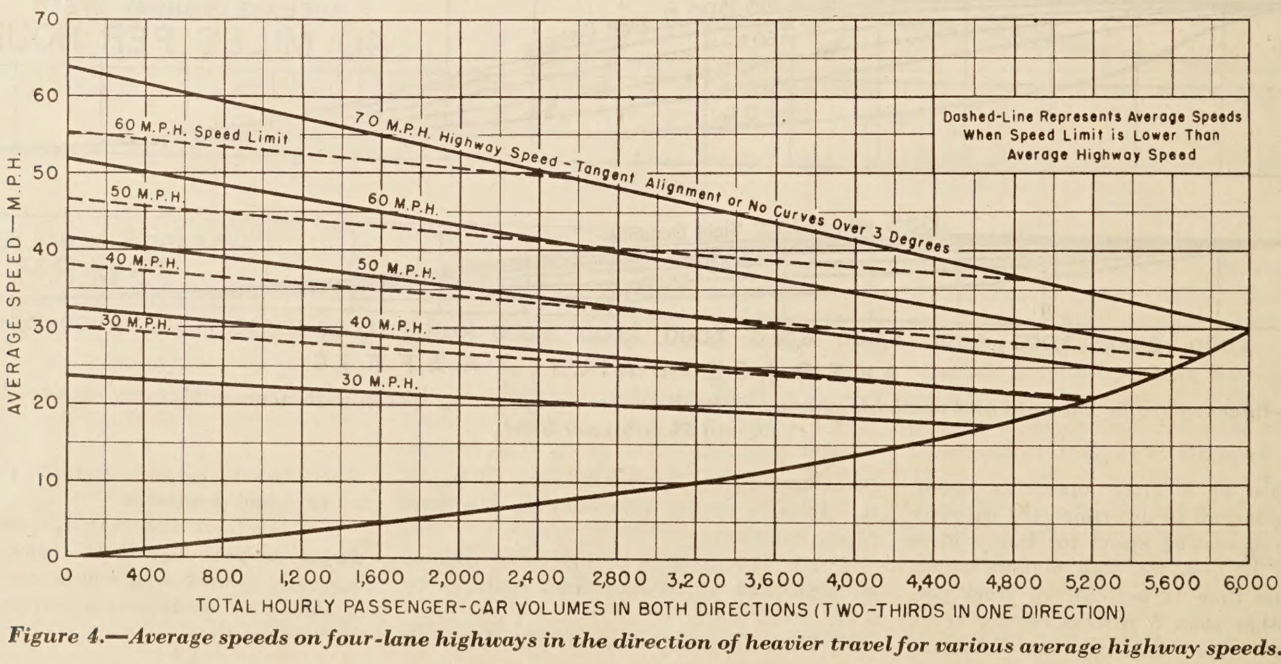


Figure 4.—Average speeds on four-lane highways in the direction of heavier travel for various average highway speeds.

5 percent trucks with a passenger-car equivaent of 2, whereas figure 4 shows hourly volımes and includes no trucks.

These two charts represent average condiions found on modern highways throughout he United States. In some areas, such as he central States where the terrain is level ınd speeds are higher than for the country as ı whole, the speeds as shown by these charts vill be somewhat low especially for the low raffic volumes. For certain other areas they nay be high, but in general any difference will ıot be great and the relative speeds for the lifferent conditions will be accurate.

The traffic volumes or capacities at a given perating speed or at a given average speed ıre shown in terms of numbers of vehicles in wo 12-foot lanes for the one direction of ravel. Daily and hourly volumes or capacties for various percentages of trucks and a ange of truck factors may be determined by tandard procedures.

The results for multilane highways as shown y figures 3 and 4 explain to a large extent the nany variations in the speed-volume relation ound by other investigators. Sometimes they ave found that an increase in the traffic volme or density results in only a very slight or o drop in speeds. This would be the case as hown by the dashed-lines in figures 3 and 4 vhen a speed limit or factors other than the raffic density are exerting a controlling influnce on vehicle speeds.

The results of still other investigators show curvilinear relation with the speeds dropping t an increasing rate as the traffic density ncreases. This would occur as the traffic volmes exceed the range within which the speed mits are effective and especially when the olumes approach possible capacities. At olumes approaching possible capacities on ıultilane facilities (above 1,500 vehicles per our per lane), the safety factor for capacity, s indicated by the distance between the upper nd lower curves of figures 3 and 4, decreases apidly with the result that a slow driver or ome other minor condition interrupting the ormal flow of traffic can cause a sudden lowdown of all vehicles with speeds decreasıg from a point on one of the higher curves f figures 3 and 4 to a point on the bottom urve, or to any intermediate point. The loser the possible capacity is approached, the reater is the possibility of such an occurrence.

The most baffling results obtained from peed-volume investigations are those which how an increase in speed with an increase in olume. Generally this occurs when a study s started during off-peak hours with light raffic and is continued through the peak or ush-hour volumes in the afternoon. As the raffic volume increases, the percentage of epeat drivers in a hurry to get home increases ith the result that speeds show little or no ecline and oftentimes increase temporarily ith the traffic volume. When capacity olumes are reached or closely approached here is then an abnormal decrease in speeds roducing the curvilinear relation between peed and traffic volume. Studies of this type o not show the true effect of increased olume or speeds, since there is a marked change in the character of traffic from off-peak to peak periods. The true effect of volume on speeds as shown in figures 3 and 4 can be obtained by simultaneous studies at different points where the geometric features of the highway are identical but the traffic volumes are different.

Information Needed for Capacity Analysis

An engineering analysis of the ability of a highway to accommodate present or estimated future traffic volumes, in accordance with prescribed standards of service in terms of operating speeds, requires the following information:

1. The type of terrain through which the highway is located.
2. The average highway speed and the frequency of occurrence of sharp curves that cause abnormally low speeds.
3. The percentage of the highway on which the passing sight distance exceeds 1,500 feet. On highways for which an operating speed of 40 miles per hour or less has been specified, the percentage of highway with an 800- to 1,000-foot sight distance is required whenever there is a low percentage of the 1,500-foot sight distance.
4. The average truck factor and the truck factor on all long or steep grades.
5. Cross-section items such as shoulder and surface type, width, and condition.

These five items were determined for all highways in West Virginia expected to carry annual volumes in excess of 1,800 vehicles per day within the next 20 years.

Type of terrain

Generally the alinement of an existing highway will be an indication of the surrounding terrain. Whether standards for level, rolling, or mountainous terrain should be applied to an existing road is largely a matter of engineering judgment. Just because the existing highway has many sharp curves and steep grades, however, does not necessarily mean that a much better alinement and profile could not be obtained in the same general vicinity at a reasonable cost with modern equipment and methods. A large part of West Virginia has terrain through which it is extremely difficult and costly to build high-speed highways of modern design.

Average highway speed

The average highway speed of each section of highway was determined by driving a passenger car over the highway at the maximum safe speed during extremely low traffic volumes to obtain a profile of the speed based on the geometric features of the highway. The safe speed was governed by sight distance, curvature, and possible marginal interferences. All speed zones and speed limits were observed. Long tangent sections of highway were recorded as having a 60-miles-per-hour highway speed even though the test car was not necessarily operated at that speed. Such sections are, however, comparatively rare in West Virginia.

This method of determining the average highway speed and obtaining a log of the sharp curves and other speed restrictions was employed because sufficiently detailed information was not available from any other source. Furthermore, this method as it was employed was sufficiently accurate and probably resulted in a more realistic appraisal than could have been obtained from detailed plans had they been available.

Passing sight distance

A second car with an accurate odometer was driven over each highway at a slow speed (about 30 m.p.h.) to determine the length and location of all sections with sight distances in excess of 1,000 feet and 1,500 feet, in lieu of more accurate and detailed sight distance information. The driver informed the passenger, who acted as the recorder, each time there was a change in the sight distance from some value below 1,000 feet or 1,500 feet to a value above 1,000 or 1,500 feet. He also informed the recorder each time the sight distance again became less than either of these values.

The recorder noted the odometer readings at these locations and at control points such as crossroads, city limits, and major structures. It was possible to check the accuracy of the driver's estimate by this procedure as each reading was recorded so that a sufficiently accurate estimate was obtained of the percentage of the highway with a sight distance in excess of 1,000 feet and the percentage in excess of 1,500 feet.

Average truck factor

Commercial vehicles with dual tires reduce the capacity of a highway in terms of vehicles per hour. In level terrain where commercial vehicles can maintain speeds that equal or approach the speeds of passenger cars, it has been found that the average dual-tired vehicle is equivalent, in a capacity sense, to 2 passenger cars on multilane highways and to 2.5 passenger cars on two-lane highways. The number of passenger cars that each dual-tired vehicle represents is termed the "truck equivalent" or the "truck factor."

The results of highway capacity studies have shown that the truck equivalent on long or steep grades increases with an increase in the difference between the normal speeds of passenger cars and the speeds of trucks. They have also shown that the truck equivalent changes very little, if at all, with a change in the percentage of trucks in the total traffic stream.[3]

Truck equivalents are normally determined by obtaining detailed information on the speeds and headways of vehicles during various traffic volumes on highways with different alinements and profiles. An average truck factor is obtained for dual-tired vehicles under each condition. If the study is of sufficient magnitude, it is possible to obtain

[3] Studies have not been conducted at locations with more than 20 percent dual-tired trucks and have been confined principally to locations with less than 10 percent of these vehicles during the periods of peak flow. Further studies may indicate that for certain conditions the truck factor does change with a change in the percentage of trucks, but as yet there is no evidence to indicate whether it increases or decreases with an increase in the number or percentage of trucks.

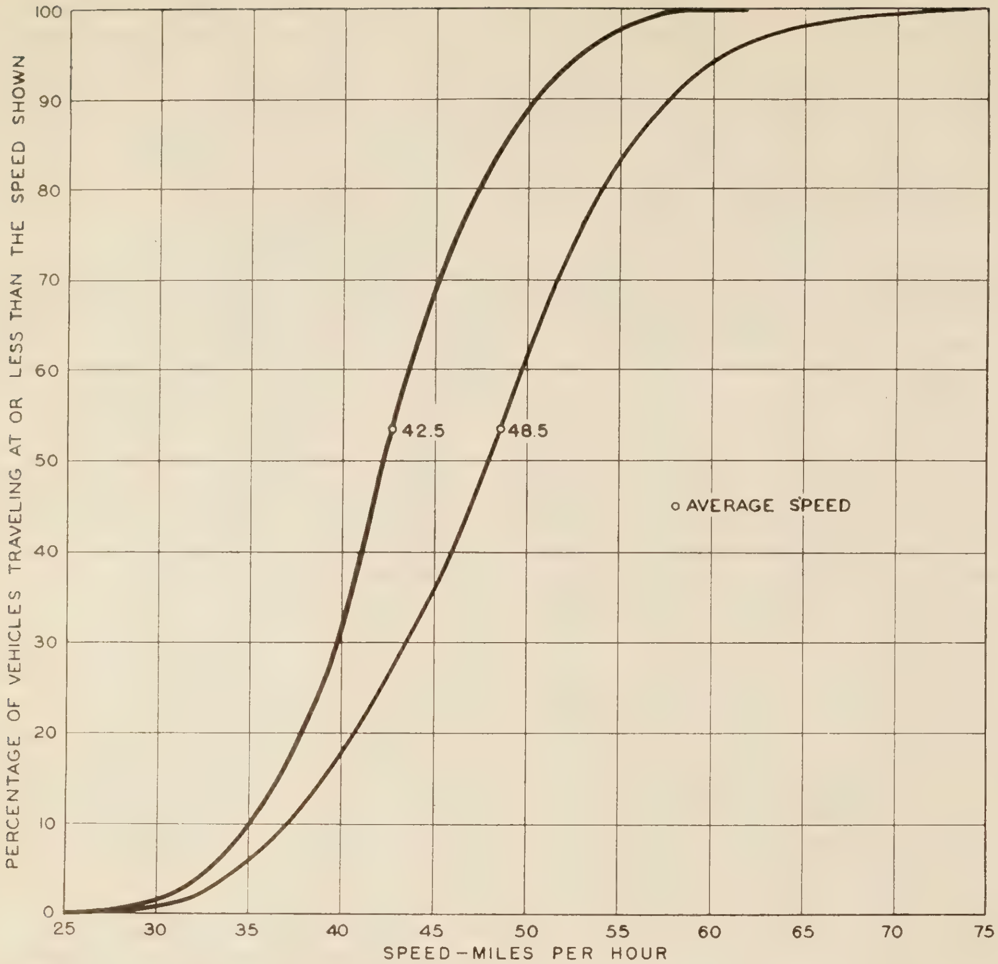


Figure 5.—Two distributions of normal passenger-car speeds used in determining truck factors (passenger-car equivalents).

Table 6.—Truck factor for various truck speeds as related to normal passenger-car speeds

Truck speed (miles per hour)	Truck factor—passenger-car equivalents		
	For average passenger-car speed of 48.5 m. p. h.[1]	For average passenger-car speed of 42.5 m. h. p.[1]	Adopted for use in West Virginia study
40	1.8	1.5	2
35	3.0	2.7	3
30	5.0	4.9	5
25	8.6	7.6	8
20	13.9	11.7	13
15	22.9	18.7	20
10	40.5	32.5	35
5	94.5	75.0	80

[1] Distributions of passenger-car speeds are shown in figure 5.

a truck factor for each type of dual-tired vehicle classified by speed groups.

The results of these studies have shown that truck factors can also be calculated with a high degree of accuracy from the separate speed distributions of passenger cars and trucks recorded during light volumes when vehicles can travel at their normal speeds. The criterion used is the relative number of passings that would be performed per mile of highway if each vehicle continued at its normal speed for the conditions under consideration. That the results from such an analysis agree with those obtained by the more painstaking methods is not surprising. It is the difference between truck speeds and passenger-car speeds on grades that causes trucks to reduce the capacity of a highway. The greater the speed difference, the greater is the reduction in capacity with a corresponding increase in the truck factor.

Table 6 shows how the truck factor varies with the truck speed for two different passenger-car speed distributions as shown in figure 5. The higher the passenger-car speeds, the higher are the truck equivalents. The factors in the right-hand column of table 6 are the rounded values used for the West Virginia study and from which figure 6 was plotted. The truck equivalent can be determined for any dual-tired vehicle by knowing its average speed under any highway condition such as a steep or long grade. The average truck factor can also be determined for any location or section of highway by knowing the average speed for all trucks if the passenger-car speeds are within the limits of those shown in figure 5. In this case, there will be a slight error if there is a wide range in the truck speeds because the curve of figure 6 is not a straight line. The error will be slight, however, for most conditions.

Control Truck Used for Obtaining Average Truck Speed

In flat or rolling terrain it is possible to conduct sufficient speed studies to determine the speeds of trucks for the typical and unusual profiles that are encountered on a

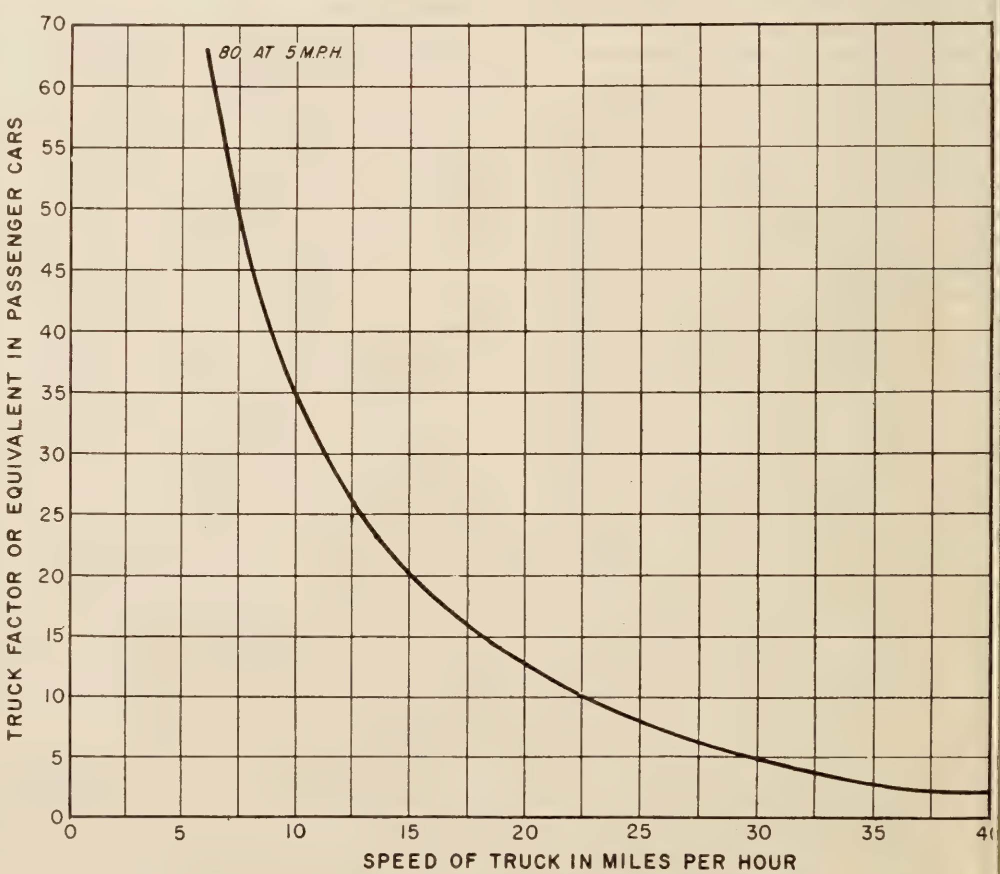


Figure 6.—Truck factors (passenger-car equivalents) for average truck speeds ranging from 5 to 40 miles per hour.

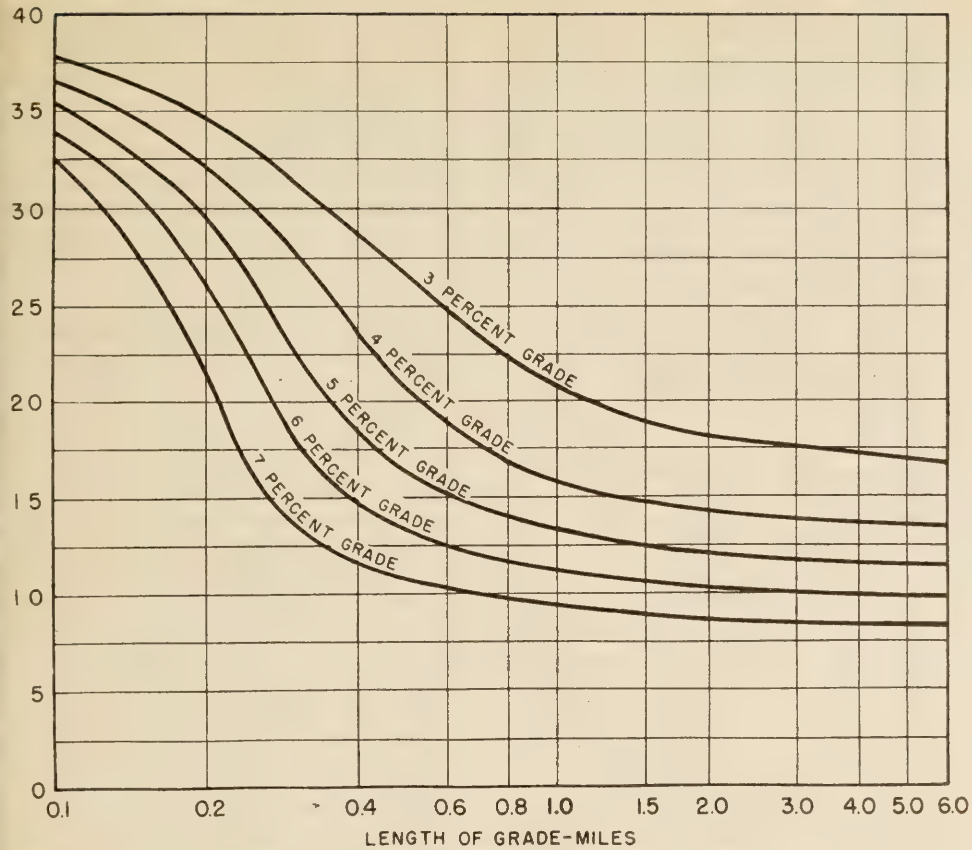


Figure 7.—Average speeds of control truck on grades ranging from 3 to 7 percent.

ighway system. In mountainous terrain, owever, this approaches an impossible task. his is especially true for West Virginia. unique method was therefore employed o obtain the average truck factor for each ection of highway and for each grade or ombination of grades on all roads in West irginia likely to carry more than 1,800 ehicles per day during the next 20 years.

The method involved the selection of a ypical truck with a typical load. This truck as driven over the highway system at its aximum safe speed consistent with normal ruck operation to obtain a continuous speed rofile. The speed of the truck and its dometer reading were recorded at the bottom nd top of each grade, at crossroads or other ontrol points, each time the gears were hifted, and each time there was a change of miles per hour in the speed of the truck. When the truck reached a crawl speed on ong grades, the crawl speed was recorded to he nearest mile per hour. The truck was perated in both directions on the more mportant roads to get a speed profile for ach direction of travel.

The control truck and its load were selected to obtain a weight-power ratio of 325 pounds per horsepower so that its effect on highway capacity would be the same as the average dual-tired vehicle. Its gross load was 40,000 pounds which is considerably lighter than the heaviest group of vehicles recorded during recent loadometer surveys, but also heavier than the average dual-tired vehicle including those with and without payloads. Since the curve in figure 6 is not a straight line, the possible speed of the control truck on an upgrade was purposely recorded somewhat lower than the average for all dual-tired trucks on the same grade. This was necessary so that the truck factor obtained for the speed of the control truck from figure 6 would equal the average factor for all trucks.

As an example, the average truck factor for speeds of 35 and 15 miles per hour is 11.5 or $(3+20)\div 2$. A truck factor of 11.5 is represented by a speed of 21 miles per hour rather than 25 miles per hour—the average of 35 and 15.

Soon after placing the control truck in operation, its speeds on hills with known gradients were checked with the performance curves for vehicles under controlled test conditions and found to be in agreement. Trial runs on the same grade were also remarkably consistent.

Speed data for trucks on grades, recorded at spot locations and also over the entire length of long grades by the stopwatch method, showed that the average truck factor was somewhat lower than the truck factor obtained by using the speed of the control truck. The difference varied from 10 to 20 percent. Since this was on the conservative side and would make a difference of less than 5 percent when used for estimating the capacities of existing roads, no adjustment or correction was made. Had it been desired to more accurately duplicate the average performance of present-day commercial vehicles as found in West Virginia, the load on the control truck should have been reduced about 5,000 pounds.

The average speeds of the control truck on 3 to 7 percent uniform grades up to 6 miles long are shown in figure 7 and table 7. Figure 8 shows the speed of the truck at any point on these grades. The speeds as shown by the solid lines are based on the assumption that the truck enters the grade at 41 miles per hour.

These curves may also be used to determine the speed reduction due to any length and steepness of grade for other approach speeds. For example, if the approach speed is 40 miles per hour (initial distance 85 feet), the speed at the top of a 4-percent grade 1,000 feet long will be 26 miles per hour (final distance 1,085 feet). Similarly, if this same grade is approached at a speed of 30 miles per hour, the speed at the top will be 17 miles per hour.

The dashed-curves emanating from 9 miles per hour show the maximum performance of vehicles when the approach speed is so low that the vehicle must accelerate to eventually reach the sustained speed. These curves show that it takes exceedingly long distances to accelerate on grades when the approach speed is below that of the sustained speed. To change the speed on a 2-percent grade from 20 miles per hour to the sustained speed of 21.5 miles per hour, an increase of only 1.5 miles per hour, the vehicle would have to travel 1,050 feet.

If needed, similar curves can be prepared for trucks with other weight-power ratios or for other entering speeds from the results of motor-vehicle performance studies conducted by the Bureau of Public Roads and others (*3–6*). This was not necessary for the West Virginia needs study because the truck was operated over all routes under consideration.

Table 7.—Average speed of typical truck entering grades at a speed of 40 miles per hour

Gradient	Average speeds on grades extending—												Sustained speed on grade	Distance required to reach sustained speed
	0.1 mile	0.2 mile	0.4 mile	0.6 mile	0.8 mile	1.0 mile	1.5 miles	2.0 miles	3.0 miles	4.0 miles	5.0 miles	6.0 miles		
Pct.	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. y.*	*M p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*M. p. h.*	*Miles*
3	37.3	34.6	28.4	24.6	21.9	20.4	18.7	17.9	17.3	16.9	16.7	16.6	16.0	0.78
4	36.1	31.7	23.4	18.5	16.6	15.7	14.6	14.1	13.6	13.4	13.3	13.2	12.8	.60
5	35.2	29.3	18.2	14.9	13.7	13.1	12.3	11.9	11.6	11.5	11.4	11.3	11.0	.37
6	34.0	25.8	14.5	12.4	11.5	11.0	10.5	10.2	10.0	9.8	9.8	9.7	9.5	.28
7	32.6	21.4	11.8	10.2	9.5	9.2	8.8	8.5	8.4	8.3	8.2	8.2	8.0	.24

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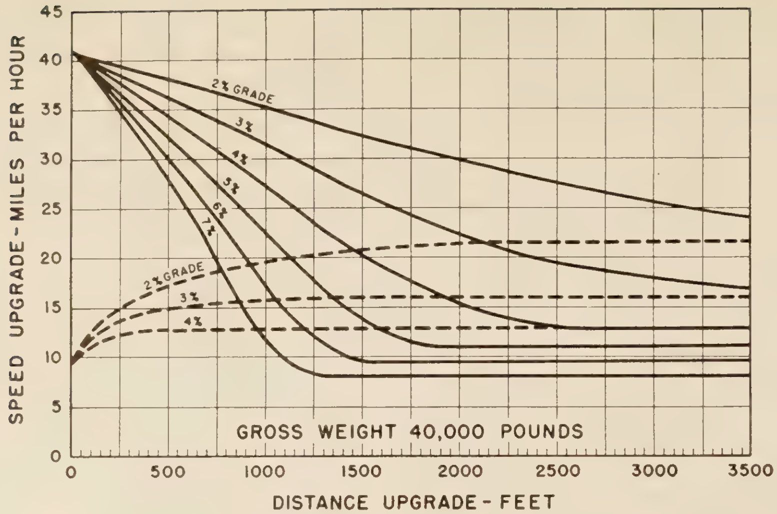

Figure 8.—Effect of length of grade on the speed of the control truck.

If the grades had been uniform and their length and gradient known, it would have been possible to determine the average truck factor by applying the data from figure 7 to figure 6. Driving the truck over the routes would have been unnecessary. This method was employed in Kentucky and Tennessee. In West Virginia, however, the needed information for the grades was not available. Furthermore, in this State there are few uniform grades. Practically all have multiple gradients for which it is possible, but rather difficult and time-consuming, to calculate truck speeds accurately. One such example is shown in figure 9. Also shown is the speed profile recorded for the control truck.

Truck Climbing Lanes Increase Capacity

Truck climbing lanes on the uphill side of long steep grades, as shown in figure 10, provide a means for improving the capacity of two-lane roads through rough or mountainous terrain. It is on a long steep grade that the greatest difference occurs between the normal speed of passenger cars and the normal speed of trucks. The need for adequate passing opportunities is therefore greatest on the long steep grades, whereas the passing opportunities are generally less than on the level sections of a two-lane highway. This results in higher truck factors and lower capacities for uphill sections of a two-lane highway than for the level sections.

Where truck climbing lanes are provided, the truck factor becomes zero and the capacity of the normal section of the two-lane highway is the same as though there were no trucks. Under certain conditions, therefore, truck climbing lanes will increase the practical capacity of an entire two-lane highway to a value higher than that for the same alinement with no grades. This is because the provision of a climbing lane reduces the average truck factor and increases the percentage of the highway on which passing maneuvers may be performed.

Climbing lanes will also increase the capacity of multilane highways. In fact, an added lane for each direction of travel over the entire length of a multilane highway may often be avoided by providing an added lane on the uphill side of long or steep grades.

The quantitative effect that trucks have on the capacity of multilane highways with long steep grades is not as well known, however, as for two-lane highways. For example, it is entirely possible that a few heavy trucks on a long steep grade of a multilane highway might have nearly as great an effect as a much larger number. The factors used at present are average values determined for less than 20 percent dual-tired vehicles—usually 5 to 10 percent.

Application of Uphill Truck Lanes

The benefit to traffic by providing an uphill truck lane at a specific location depends upon the following factors: (1) traffic volume, (2) percentage of trucks, (3) length and steepness of grade, and (4) availability of passing sight distance.

The information in table 8 offers some guidance for the application of climbing lanes. The fourth column in this table, for example, shows the lengths of grade for an average truck speed of 34 miles per hour or a truck factor of 3.0. At this average speed, even

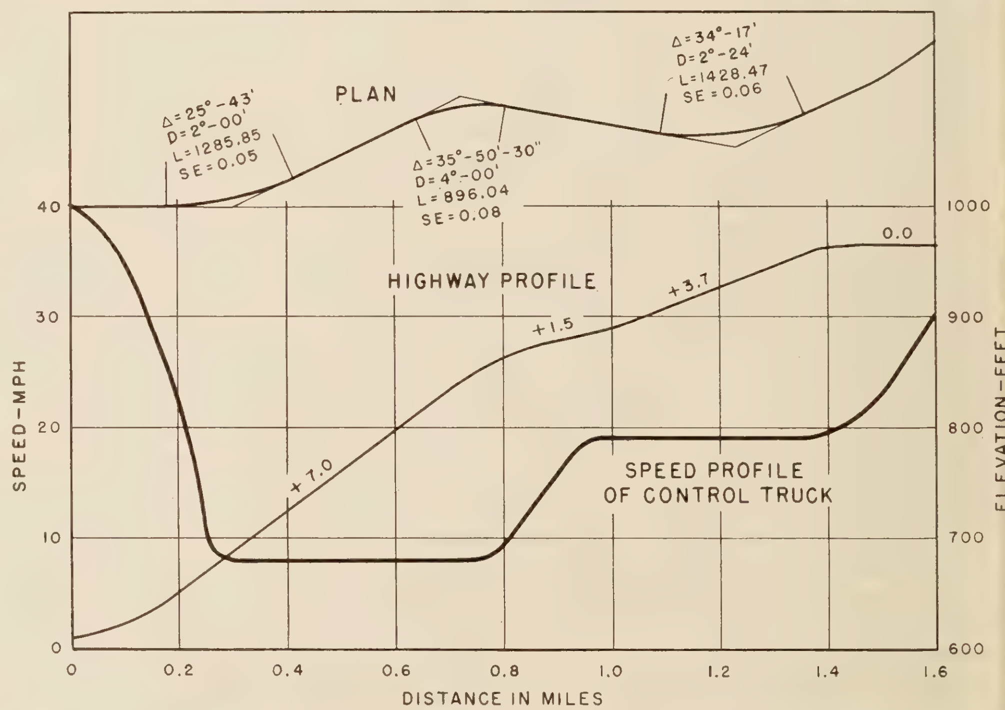

Figure 9.—Speed profile of the control truck.

Figure 10.—Truck climbing lane on U. S. Route 40, south of Middletown, Md.

Table 8.—Speed characteristics of control truck on upgrades, when entering grade from level section at 40 miles per hour

Gradient	Crawl speed[1]		Distance upgrade for an average speed of—		
	Velocity	Distance upgrade	34 m.p.h. or a truck factor of 3.0	27 m.p.h. or a truck factor of 6.5	19 m.p.h. or a truck factor of 13.8
Pct.	*M.p.h.*	*Ft.*	*Ft.*	*Ft.*	*Ft.*
3	16.0	4,000	1,100	2,000	6,600
4	12.8	2,600	800	1,500	3,000
5	11.0	1,800	600	1,200	2,000
6	9.5	1,500	500	1,000	1,500
7	8.0	1,300	400	800	1,200

[1] Speed which truck can maintain indefinitely.

though about half of the trucks will be traveling at somewhat lower speeds, the speeds of passenger cars will not be affected sufficiently to greatly inconvenience the drivers. At traffic volumes approaching practical capacities for level sections of two-lane highways, few passenger cars will overtake a truck on grades that are shorter than those shown in column 4. For those that do, the necessary reduction in speed and the lost time in reaching the top of the grade when the passing sight distance is restricted will not be appreciably greater than commonly necessary due to oncoming traffic on straight level sections. Truck climbing lanes cannot be justified, therefore, on grades shorter than those shown in the fourth column of table 8.

The fifth and sixth columns of table 8 show lengths of grade on which there is the same relative need for a truck climbing lane. With a given traffic volume, for example, there is the same need for a climbing lane on a 3-percent grade 2,000 feet long as on a 7-percent grade 800 feet long.

The capacities of two-lane highways on grades with and without truck climbing lanes are shown in table 9 for the conditions applicable to West Virginia. The various groups shown for the length of grade in the second column are purely arbitrary with the exception of the shortest length shown for each gradient. The grades could have been divided into a larger or smaller number of length groups with corresponding changes in the average annual traffic volumes. The number of groups that have been used are believed to be consistent with the accuracy justified by the analyzed data.

Table 9 is based on the assumption that each climbing lane will be continuous from a point near the bottom of the grade to a point beyond the top of the grade where the sight distance becomes unrestricted and truck speeds again approach those of passenger cars. All steep grades of equal gradient longer than 4,000 feet have the same capacities. Prior to traveling 4,000 feet upgrade, most trucks will have reached their crawl speeds.

For certain traffic and terrain conditions on exceedingly long grades, the use of passing bays may be an adequate and a more feasible solution than a continuous climbing lane (*5*). With passing bays, the capacity of a two-lane road would be greater than without the passing bays and, for certain conditions, might equal the capacities shown in table 9 for the two-lane roads with a truck lane. The maximum capacities with continuous truck lanes are actually higher than most of the values in table 9. It was assumed that the capacity on a grade with a truck lane could not exceed the capacity of a two-lane level section. The capacity with a truck lane falls below the capacity of a level section only on the long grades over 5 percent where downhill speeds of trucks traveling in the lower gears affect capacities.

Application to Capacity Determinations

The tables and charts that have been presented are the basic information needed for capacity determinations in connection with the West Virginia needs studies. From this information, an almost unlimited number of special tables and charts can be prepared for specific conditions in either West Virginia or other States. The data can also be applied in many different ways as will be explained by the applications made for the West Virginia, Kentucky, and Tennessee studies.

In order to determine the highway needs in West Virginia, it was necessary to have a vast amount of information concerning the roads and the traffic using them. For the capacity determination with which this report is concerned, only the factors that have been previously discussed were needed. Their effect on the capacities of two-lane roads can be determined from tables 1, 5, and 9, and figure 6. Figure 7 was also needed for the Kentucky and Tennessee studies since a control truck was not used to determine the truck factors in these States.

It is important that the conditions be similar over a length of highway for which a capacity determination is made. Section limits, for this reason, were usually defined by urban limits; or by a change in the traffic volume, surface width, average highway speed, or type of terrain; or by a marked change in the percentage of highway with a 1,500-foot passing sight distance. In addition, a county line was the end of one section and the beginning of another.

Application to West Virginia highways

Five typical sections analyzed during the West Virginia studies will illustrate the procedures used to apply the capacity information. The basic information and the resulting calculations for each of these sections are shown in table 10.

Section 1 as shown in table 10 is located on U. S. Route 60 about 20 miles west of Charleston in Putnam County. It is 6.4 miles long with a 26-foot pavement in rolling terrain. It has an excellent passing distance as compared with most West Virginia roads, since 59 percent of its length has a sight distance in excess of 1,500 feet. The average highway speed is 65 miles per hour, and the generally flat profile results in a truck equivalent of only 2. The average daily traffic volume was 5,500 in 1955 with a design-hour factor of 12 percent of the ADT having 7 percent trucks.

The capacity of this section is 5,800 vehicles daily at an operating speed of 45–50 miles per hour, or 7,150 vehicles daily at a tolerable operating speed of 40–45 miles per hour. As U. S. Route 60 is one of the most important highways in the State, it is desirable to provide conditions conducive to a high operating speed.

For an operating speed of 45–50 miles per hour the existing traffic volume is practically equal to the capacity of the section. As it would be impractical to attempt to increase the capacity of the existing road by improving passing sight distances, the only recourse to accommodate expected future traffic volumes is to add additional lanes by constructing

Table 9.—Capacities of two-lane highways on grades carrying 5-percent truck traffic, based on a 30th highest hour of 12 percent of the average annual traffic volume

Gradient	Length of grade	Average annual traffic volumes							
		Average highway speed, 70 m. p. h.; operating speed, 50–55 m. p. h.		Average highway speed, 60–70 m. p. h.; operating speed, 50 m. p. h.		Average highway speed, 60–70 m. p. h.; operating speed, 45–50 m. p. h.		Average highway speed, 50–70 m. p. h.; operating speed, 40–55 m. p. h.	
		Without truck lane	With truck lane	Without truck lane	With truck lane	Without truck lane	With truck lane	Without truck lane	With truck lane
Pct	*Ft.*	*V. p. d.*	*V. p. d.*	*V. p. d.*	*V. p. d.*	*V. p. d.*	*V. p. d.*	*V. p. d.*	*V. p. d.*
3	1,100–2,000	4,300	4,700	4,850	5,500	5,150	6,000	6,500	7,000
	2,000–4,000	3,850	4,700	4.300	5,500	4,550	6,000	6,250	7,000
	Over 4,000	3,500	4,700	3,800	5,500	4,050	6,000	5,550	7,000
4	800–1,500	4,300	4,700	4,850	5,500	5,150	6,000	6,500	7,000
	1,500–3,000	3,850	4,700	4,200	5,500	4,400	6,000	5,800	7,000
	3,000–4,000	3,400	4,700	3,750	5,500	4,000	6,000	5,450	7,000
	Over 4,000	3,200	4,700	3,400	5,500	3,800	6,000	5,100	7,000
5	600–1,200	4,300	4,700	4,850	5,500	5,150	6,000	6,500	7,000
	1,200–2,000	3,500	4,700	3,800	5,500	4,050	6,000	6,150	7,000
	2,000–4,000	3,200	4,700	3,700	5,500	3,950	6,000	5,700	7,000
	Over 4,000	2,800	4,700	3,250	5,500	3,500	6,000	4,800	7,000
6	500–1,000	4,300	4,700	4,850	5,500	5,150	6,000	6,500	7,000
	1,000–1,500	3,500	4,700	3,800	5,500	4,050	6,000	6,150	7,000
	1,500–4,000	3,050	4,200	3,550	5,200	3,800	6,000	5,350	7,000
	Over 4,000	2,550	3,600	2,950	4,000	3,200	4,200	4,500	5,800
7	400– 800	4,300	4,700	4,850	5,500	5,150	6,000	6,500	7,000
	800–1,200	3,500	4,700	3,800	5,500	4,050	6,000	5,550	7,000
	1,200–2,500	2,900	4,200	3,400	5,000	3,650	6,000	5,100	7,000
	2,500–4,000	2,600	3.600	3,000	4,000	3,300	4,200	4,600	5,800
	Over 4,000	2,000	3,000	2,400	3,400	2,650	3,600	3,700	4,600

Table 10.—Capacity analysis of typical highway sections in West Virginia

	Section No. 1	Section No. 2	Section No. 3	Section No. 4	Section No. 5
KNOWN CONDITIONS					
Route designation	U. S. 60	U. S. 21	U. S. 60	U. S. 50	U. S. 50
Length of section ... miles	6.37	6.21	7.00	4.13	11.46
Type of terrain	Rolling	Rolling	Rolling	Rolling	Rolling
Average highway speed ... m. p. h.	65	40	53	57	53
1,500-foot sight distance, percent available	59	2	10	9	15
Surface width ... ft.	26	18	22	22	20
30th-highest-hour factor ... pct.	12	12	12	13	14
Truck speed ... m. p. h.	40	32	29	25	24
Commercial vehicles ... pct.	7	5	5	5	5
1955 average daily traffic	5,500	2,300	2,400	2,400	2,600
Long grades	None	None	(1)	(1)	(1)
Sharp curves	None	None	(2)	None	(2)
DETERMINED VALUES					
Tolerable operating speed [3] ... m. p. h.	40–45	40–45	40–45	40–45	40–45
Tolerable design speed [3] ... m. p. h.	50	50	50	50	50
Tolerable capacity [4] ... ADT	7,300	None	3,070	3,800	3,405
Width factor	1.00	.70	.86	.86	.77
30th-hour factor	1.00	1.00	1.00	.92	.86
Truck equivalent	2.0	4.0	5.0	8.0	9.0
Truck factor	.98	.91	.88	.78	.75
Corrected tolerable capacity [5] ... ADT	7,150	None	2,300	2,350	1,700

[1] Section 3, 0.40 mile at 8 percent; section 4, 0.50 mile at 7.5 percent, 0.35 mile at 4 percent, 0.60 mile at 2.5 percent; section 5, 0.5 mile at 6.5 percent, 1.2 miles at 5 percent, 2.5 miles at 2.5 percent.
[2] Section 3, three curves at 30, 35, and 45 miles per hour, and two curves at 50 miles per hour; section 5, one curve at 40, one at 45, and three at 50 miles per hour.
[3] Taken from table 1.
[4] Taken from table 5.
[5] Computed as follows: 105+(100−percentage of commercial vehicles)+(percentage of commercial vehicles × truck equivalent).

another one-way roadway and using the existing lanes for the other direction of travel.

Section 2 on U. S. Route 21 in Jackson County is also on one of the more important roads in the State, although the traffic volume is not high. The 6.2-mile section is located in rolling terrain about 20 miles north of Charleston. Both the alinement and profile are poor, resulting in a low design speed and limited sight distance. Pavement width is 18 feet and the truck equivalent is 4. The traffic volume during 1955 was 2,300 vehicles per day with the design-hour factor of 12 percent of the ADT with 5 percent trucks.

The road as it exists today does not meet the tolerable standards for this class of highway. Since its average highway speed is only 40 miles per hour, it cannot carry traffic at the tolerable speed of 50 miles per hour during low volumes nor at 40 to 45 miles per hour during the 30th highest hourly volume of the year. It therefore has no capacity for these speeds.

Some improvement in alinement could be made to increase the average highway speed and the amount of passing sight distance. By providing a 1,500-foot sight distance over 10 percent of the length and raising the average highway speed to 45 miles per hour, the capacity would be increased to 2,200 vehicles per day at an operating speed of 35–40 miles per hour, or to 1,050 vehicles per day at a 40- to 45-miles-per-hour operating speed.

Widening the entire section to 24 feet would increase the capacities at the 35- to 40- and 40- to 45-miles-per-hour operating speeds to 3,100 and 1,600 vehicles per day, respectively.

Since the tolerable operating speed for this highway is 40–45 miles per hour, the 35- to 40-miles-per-hour operating speed would be inadequate and undesirable. The capacity at a minimum desirable operating speed with the alinement and sight distances improved to the extent possible on the existing location is still considerably less than the existing traffic volume. The conclusion is, therefore, that the only lasting solution is a complete redesign of the highway.

Section 3 is located on U. S. Route 60 in Greenbrier County about 100 miles east of Charleston. The terrain is rolling over this 7.0-mile section, the average highway speed is 53 miles per hour, about 10 percent of the highway has a 1,500-foot sight distance, and the truck equivalent is 5. Tolerable operating speed for this highway is 40–45 miles per hour. At this speed the capacity of the section is 2,300 vehicles per day.

Several possibilities are available for increasing the capacity of the section, including removal of some or all of the five substandard curves, the addition of truck lanes on grades, and minor improvements in the sight distance by removal of trees, daylighting curves, etc.

Reducing curvatures would increase the average highway speed to about 55 miles per hour, resulting in a tolerable capacity of about 2,600 vehicles per day. Passing sight distance might be increased an additional 5 percent by miscellaneous measures, such as brush removal, curve daylighting, etc. This would further increase capacity to about 2,800 vehicles per day.

The next alternative is the provision of truck lanes. The existing grades would require about one mile of truck lanes to be added, resulting in a decrease in the overall truck equivalent from 5 to 3. Minor improvement in the alinement would also provide additional passing sight distance so that a 1,500-foot sight distance would be available over approximately 20 percent of the highway. All these improvements would increase the capacity of the highway to about 3,380 vehicles per day at an operating speed of 40–45 miles per hour. At the normal rate of traffic growth, this volume would not be exceeded for a period of 6 or 7 years. Thereafter it would be necessary to undertake major changes in the alinement or to provide a four-lane highway in order to maintain the desired operating speed.

Section 4 has 11-foot traffic lanes and is located on U. S. Route 50 in Wood County. It is a 4.1-mile section through rolling terrain. The alinement is fairly good since the average highway speed is 57 miles per hour and 9 percent of the highway has a 1,500-foot sight distance. The traffic volume in 1955 was 2,400 vehicles per day with a design-hour factor of 13 percent of the ADT including 5 percent trucks.

For the tolerable operating speed of 40–45 miles per hour, the capacity is 2,350 vehicles per day. This is slightly lower than the present volume. Building a truck lane one mile long on a critical grade would reduce the truck equivalent to 3 and would increase the 1,500-foot passing sight distance from 9 percent to about 18 percent of the length. As a result the capacity would be increased to 3,250 vehicles per day at an operating speed of 40–45 miles per hour. Some additional 1,500-foot sight distance could be obtained by increasing the view on the inside of several curves by simply removing the obstructions such as brush and low banks on the right-of-way. When the obstruction is off the right-of-way, additional right-of-way must be purchased or an agreement reached with the property owner to keep it cleared. An additional 5 percent of 1,500-foot sight distance can be obtained in this manner. This would increase the capacity at the desired operating speed to 3,450 vehicles per day, which represents an increase of nearly 60 percent over the present traffic volume, or to approximately the volume expected in 1970.

Section 5 is located on U. S. Route 50 in Hampshire County in the northeastern part of the State. The section is 11.5 miles long with uniform design characteristics in the rolling terrain. The average highway speed is 53 miles per hour, the surface width is 20 feet, and 15 percent of the highway has a 1,500-foot sight distance. The truck equivalent is 9. The present ADT is 2,600 per day with a design-hour factor of 14 percent including 5 percent trucks. Under these conditions, the capacity at an operating speed of 40–45 miles per hour is 1,700 vehicles per day.

Several possibilities exist for improving the capacity. These include reducing the sharpness of five substandard curves, widening the surface, the addition of truck lanes on grades, and minor improvement in the sight distance. Widening from 20 to 24 feet would increase the capacity to 2,200 vehicles per day. Removal of the substandard curves will increase the average highway speed to about 55 miles per hour and would increase the passing sight distance 1 to 2 percent. These improvements, including the widening, would result in increasing the capacity to 2,550 vehicles per day.

The addition of 2½ miles of truck lanes would increase the sections on which passings could be performed to about 25 percent of the highway and reduce the truck equivalent to 3. The total resulting capacity would be 3,600 vehicles per day or 38 percent above the present volume.

These five examples are rather typical of the way the capacity information was applied in West Virginia to determine highway sufficiency. Its use was found especially helpful in pointing out the changes that could be made to improve capacity. Altering some highway features will have little effect on the capacity at a desired operating speed, while others, such as the provision of truck lanes and substantially improving the passing sight distances, will have a major effect.

Application to Kentucky and Tennessee highways

The principles employed for capacity determinations in West Virginia have general application wherever curvatures and grades create special highway capacity problems. This was the case throughout most of Kentucky and Tennessee where highway needs studies were started during the period that the West Virginia study was being completed.

The two special features needed in the refinement of the capacity analysis, which were the average highway speed and the truck equivalent, could have been obtained in the same manner as described for West Virginia. Utilizing the experience gained in the West Virginia study, however, it was found desirable and more feasible to derive these data from existing records rather than from test vehicle operation.

Kentucky and Tennessee lacked data on actual truck operations which would be consistent with probable future conditions. Following many years of severe restrictions on truck sizes and weights, Tennessee had just revised its law so as to be in substantial agreement with AASHO recommendations. Truck operations, however, had not as yet changed to conform with the higher limits. Kentucky still retained its low limits, but it was anticipated that a more realistic position would be adopted—as it was in 1956—bringing that State in line with Tennessee and the other States. Without actual data on vehicle weights for the revised weight limits, it was assumed that future conditions in Kentucky and Tennessee would be similar to those on which the West Virginia study was based.

In both Kentucky and Tennessee, geometric design data were available, mile by mile, in the State Highway Planning Division records, or were easily obtainable from plans. Thus actual curvature was known, and curve lengths could be obtained or sampled from the plans. In both States, the gradient and the length of the grades on each section of highway were available from the plans. This was not the case for most roads in West Virginia.

Alternate method used for determining average highway speed in Kentucky and Tennessee.—Operation of a test car, as in West Virginia, accounted for several factors that would affect the average highway speed, but horizontal curvature was by far the most significant. From available data, therefore, it was possible to approximate the average highway speed of control sections in the other States by concentrating the analysis on the combined effect of horizontal curves and tangents.

Vehicle speeds are affected ahead and beyond a curve for a distance which varies with the degree of curvature. That is, a vehicle on a tangent approaching a sharp curve must begin to slow down before reaching the curve in order to reduce its speed to the allowable speed on the curve. After traveling around the curve, additional time and distance are required to accelerate back to the normal tangent speed. It was therefore necessary to determine the following information for each section of highway requiring a separate capacity analysis:

1. The possible safe speed, or design speed, of each curve.

2. The length of each curve.

3. The distance before and after each curve that the speed was affected, together with the average speed while decelerating and accelerating.

4. The average speed weighted by the length of the tangents, the curves, and by the deceleration and acceleration distances. This speed was used as the average highway speed.

The safe speeds for curves of various degrees (or radii) were determined from the tables in the AASHO policy on *Geometric Design of Rural Highways* (1). The length of each curve was obtained from the highway plans or from planning survey information. Comfortable rates of acceleration and deceleration as shown in the AASHO policies were used to determine the length of speed transitions between the curves and tangents.

A special study conducted by sampling the curves on level sections from Kentucky highway plans showed that, regardless of curvature, the average total effect of a curve on the speed of a vehicle was equivalent to a travel distance of about 800 feet at the safe speed for the curve. For example, the 9-degree curves good for a design speed of 45 miles per hour had an average length of 667 feet. Decelerating and accelerating from the 65 miles per hour tangent speed required a total of 485 feet. On an average, a vehicle would be affected for a total distance of 1,152 feet but the time lost was the same as if the speed was 45 miles per hour for 915 feet and the tangent speed was 65 miles per hour on the rest of the section. Likewise for the 40-degree curves, the equivalent distance at 20 miles per hour was 691 feet. The equivalent distances varied from curve to curve but the average was 780 feet with values much greater or less than the average being comparatively rare. An equivalent length of 800 feet or 0.15 mile for all curves was therefore used to determine the average highway speeds for the highway sections in Kentucky and Tennessee.

Tangent sections and curves as sharp as 3 degrees were assumed to have a highway speed of 70 miles per hour if there were no curves as sharp as 4 degrees on the highway. If any curves on the highway were as sharp as 4 degrees, the tangent sections and the curves of 4 degrees or flatter were assumed to have a highway speed of 65 miles per hour. These assumptions are in accordance with the AASHO definition of design speed as related to the travel speeds found on main rural highways during low traffic densities.

Table 11.—Illustration of method used in estimating the average highway speed of a two-lane highway

Curvature	Safe speed	Number of curves	Total length	Product of columns 2 and 4
Degrees	*M. p. h.*		*Miles*	
6	55	1	0.15	8.25
10	43	2	.30	12.90
12	40	8	1.20	48.00
20	35	4	.60	21.00
30	25	1	.15	3.75
0	65	0	7.60	494.00
Total	------	-------	10.00	587.90
Average highway speed.....587.90÷10.00=59 m. p. h.				

Table 11 illustrates the method used in estimating the average highway speed of a two-lane section of highway 10 miles long. If weighted by travel time the average highway speed would be 56 miles per hour. Within the limits of reasonable accuracy, however, either method should be satisfactory. For the needs studies conducted in Ontario, Canada, the weighting to obtain the average highway speed was done on the basis of time involved rather than length.

Method used for determining truck equivalent in Kentucky and Tennessee.—The first section of this article calls attention to the fact that driving a test truck to establish a speed profile would be unnecessary if gradient and length were known, since available test data are adequate to establish truck speeds on known grades (fig. 8).

With grade data available in Kentucky and Tennessee, the truck equivalent in terms of passenger cars, for capacity computations, was determined from figures 7 and 6, in that order.

It was first assumed that the entering speed of trucks approaching a grade was 40 miles per hour. It is recognized that momentum from downgrades and actual level speeds may frequently be greater, but in the mountainous terrain where this analysis was especially pertinent, horizontal curvature is such that higher speeds are seldom encountered. For example, the speed profile of the test truck on U. S. Route 50 in West Virginia shows a maximum of only 45 miles per hour for short distances at only three locations in a 50-mile section.

It was also assumed for the purposes of this study that the average truck speed was 40 miles per hour on level terrain, on all grades of less than 3 percent, and on grades of 3 percent less than 500 feet long. For all other grades, the average truck speed was determined from curves in figure 7 for each grade or average compound grade in one direction only.

For the control section or a long subsection, the average truck speed was determined by weighting by distance the speeds on level terrain and the several grades. Finally, the

(Continued on page 44)

Table A.—Tolerable capacities of existing two-lane highways located in flat terrain, assuming an operating speed of 45–50 miles per hour and 5-percent truck traffic during 30th highest hour

Percentage of highway with passing sight distance of—		Average annual daily traffic volumes of two-lane highways with—															
		12-foot traffic lanes and passenger-car speeds at low volume of—				11-foot traffic lanes and passenger-car speeds at low volume of—				10-foot traffic lanes and passenger-car speeds at low volume of—				9-foot traffic lanes and passenger-car speeds at low volume of—			
1,500 feet	1,000 feet	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.
100	100	7,100	7,100	6,600	5,650	6,100	6,100	5,700	4,850	5,450	5,450	5,100	4,350	4,950	4,950	4,600	3,950
80	90	6,800	6,400	5,750	5,050	5,850	5,500	4,950	4,350	5,250	4,950	4,450	3,900	4,750	4,500	4,000	3,550
60	80	6,400	5,550	4,800	4,000	5,500	4,750	4,150	3,450	4,950	4,250	3,700	3,100	4,500	3,900	3,350	2,800
40	70	5,750	4,600	3,900	2,800	4,950	3,950	3,350	2,400	4,450	3,550	3,000	2,150	4,000	3,200	2,750	1,950
20	60	4,900	3,750	2,800	2,000	4,200	3,200	2,400	1,700	3,750	2,900	2,150	1,550	3,450	2,600	1,950	1,400
0	50	3,800	2,800	2,000	1,250	3,250	2,400	1,750	1,050	2,900	2,150	1,550	950	2,650	1,950	1,400	850

Table B.—Tolerable capacities of existing two-lane highways located in flat terrain, assuming an operating speed of 40–45 miles per hour and 5-percent truck traffic during 30th highest hour

Percentage of highway with passing sight distance of—		Average annual daily traffic volumes of two-lane highways with—															
		12-foot traffic lanes and passenger-car speeds at low volume of—				11-foot traffic lanes and passenger-car speeds at low volume of—				10-foot traffic lanes and passenger-car speeds at low volume of—				9-foot traffic lanes and passenger-car speeds at low volume of—			
1,500 feet	1,000 feet	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.
100	100	8,450	8,050	7,450	6,550	7,250	6,900	6,400	5,650	6,500	6,200	5,750	5,050	5,900	5,650	5,200	4,600
80	87	7,700	7,300	6,700	5,800	6,600	6,300	5,750	5,000	5,950	5,600	5,150	4,450	5,400	5,100	4,700	4,050
60	76	6,800	6,450	5,700	4,850	5,850	5,550	4,900	4,150	5,250	4,950	4,400	3,750	4,750	4,500	4,000	3,400
40	64	5,900	5,300	4,600	3,700	5,050	4,550	3,950	3,200	4,550	4,100	3,550	2,850	4,150	3,700	3,200	2,600
20	52	4,950	4,100	3,200	2,200	4,250	3,550	2,750	1,900	3,800	3,150	2,450	1,700	3,450	2,850	2,250	1,550
0	40	3,950	2,700	1,950	1,250	3,400	2,300	1,700	1,100	3,050	2,100	1,500	950	2,750	1,900	1,350	900

Table C.—Tolerable capacities of existing two-lane highways located in rolling terrain, assuming an operating speed of 45–50 miles per hour and 5-percent truck traffic during 30th highest hour

Percentage of highway with passing sight distance of—		Average annual daily traffic volumes of two-lane highways with—															
		12-foot traffic lanes and passenger-car speeds at low volume of—				11-foot traffic lanes and passenger-car speeds at low volume of—				10-foot traffic lanes and passenger-car speeds at low volume of—				9-foot traffic lanes and passenger-car speeds at low volume of—			
1,500 feet	1,000 feet	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.
100	100	6,500	6,500	6,050	5,150	5,600	5,600	5,200	4,450	5,000	5,000	4,650	3,950	4,550	4,550	4,250	3,600
80	87	6,200	5,850	5,250	4,600	5,350	5,050	4,500	3,950	4,750	4,500	4,050	3,550	4,350	4,100	3,700	3,200
60	76	5,850	5,050	4,400	3,650	5,050	4,350	3,800	3,150	4,500	3,900	3,400	2,800	4,100	3,550	3,100	2,550
40	64	5,250	4,200	3,550	2,550	4,500	3,600	3,050	2,200	4,050	3,250	2,750	1,950	3,700	2,950	2,500	1,800
20	52	4,500	3,400	2,550	1,800	3,850	2,950	2,200	1,550	3,450	2,600	1,950	1,400	3,150	2,400	1,800	1,250
0	40	3,450	2,550	1,850	1,150	3,000	2,200	1,600	1,000	2,650	1,950	1,400	900	2,400	1,800	1,300	800

Table D.—Tolerable capacities of existing two-lane highways located in rolling terrain, assuming an operating speed of 40–45 miles per hour and 5-percent truck traffic during 30th highest hour

Percentage of highway with passing sight distance of—		Average annual daily traffic volumes of two-lane highways with—															
		12-foot traffic lanes and passenger-car speeds at low volume of—				11-foot traffic lanes and passenger-car speeds at low volume of—				10-foot traffic lanes and passenger-car speeds at low volume of—				9-foot traffic lanes and passenger-car speeds at low volume of—			
1,500 feet	1,000 feet	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.
100	100	7,700	7,350	6,800	6,000	6,600	6,300	5,850	5,150	5,950	5,650	5,250	4,600	5,400	5,150	4,750	4,200
80	85	7,050	6,650	6,100	5,300	6,050	5,700	5,250	4,550	5,450	5,100	4,700	4,100	4,950	4,650	4,250	3,700
60	72	6,200	5,900	5,200	4,400	5,350	5,050	4,450	3,800	4,750	4,550	4,000	3,400	4,350	4,150	3,650	3,100
40	58	5,400	4,850	4,200	3,400	4,650	4,150	3,600	2,900	4,150	3,750	3,250	2,600	3,800	3,400	2,950	2,400
20	44	4,500	3,750	2,900	2,000	3,850	3,200	2,500	1,700	3,450	2,900	2,250	1,550	3,150	2,600	2,050	1,400
0	30	3,600	2,450	1,800	1,150	3,100	2,100	1,550	1,000	2,750	1,900	1,400	900	2,500	1,700	1,250	800

Table E.—Tolerable capacities of existing two-lane highways located in rolling terrain, assuming an operating speed of 35–40 miles per hour and 5-percent truck traffic during 30th highest hour

Percentage of highway with passing sight distance of—		Average annual daily traffic volumes of two-lane highways with—															
		12-foot traffic lanes and passenger-car speeds at low volume of—				11-foot traffic lanes and passenger-car speeds at low volume of—				10-foot traffic lanes and passenger-car speeds at low volume of—				9-foot traffic lanes and passenger-car speeds at low volume of—			
1,500 feet	1,000 feet	55 M. p. h.	50 M. p. h.	45 M. p. h.	40 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	40 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	40 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	40 M. p. h.
100	100	9, 050	8, 750	8, 300	7, 200	7, 800	7, 500	7, 150	6, 200	7, 000	6, 750	6, 400	5, 550	6, 350	6, 100	5, 800	5, 050
80	83	8, 200	8, 000	7, 500	6, 500	7, 050	6, 900	6, 450	5, 600	6, 300	6, 150	5, 800	5, 000	5, 750	5, 600	5, 250	4, 550
60	67	7, 400	7, 250	6, 600	5, 500	6, 350	6, 250	5, 700	4, 750	5, 700	5, 600	5, 100	4, 250	5, 200	5, 100	4, 620	3, 850
40	51	6, 500	6, 350	5, 600	4, 300	5, 600	5, 450	4, 800	3, 700	5, 000	4, 900	4, 300	3, 300	4, 550	4, 450	3, 920	3, 000
20	36	5, 750	5, 350	4, 100	2, 850	4, 950	4, 600	3, 500	2, 450	4, 400	4, 100	3, 150	2, 200	4, 000	3, 750	2, 900	2, 000
0	20	4, 900	3, 500	2, 250	1, 400	4, 200	3, 000	1, 950	1, 200	3, 800	2, 700	1, 750	1, 100	3, 450	2, 450	1, 600	1, 000

Table F.—Tolerable capacities of existing two-lane highways located in mountainous terrain, assuming an operating speed of 45–50 miles per hour and 5-percent truck traffic during 30th highest hour

Percentage of highway with passing sight distance of—		Average annual daily traffic volumes of two-lane highways with—															
		12-foot traffic lanes and passenger-car speeds at low volume of—				11-foot traffic lanes and passenger-car speeds at low volume of—				10-foot traffic lanes and passenger-car speeds at low volume of—				9-foot traffic lanes and passenger-car speeds at low volume of—			
1,500 feet	1,000 feet	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	70 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.
100	100	5, 500	5, 500	5, 150	4, 400	4, 750	4, 750	4, 450	3, 800	4, 250	4, 250	3, 950	3, 400	3, 850	3, 850	3, 600	3, 100
80	87	5, 300	5, 000	4, 450	3, 900	4. 550	4, 300	3, 850	3, 350	4, 100	3, 850	3, 450	3, 000	3, 700	3, 500	3, 100	2, 750
60	76	5, 000	4, 300	3, 750	3, 100	4, 300	3, 700	3, 200	2, 650	3, 850	3, 300	2, 900	2, 400	3, 500	3, 000	2, 600	2, 150
40	64	4, 450	3, 550	3, 050	2, 200	3, 850	3, 050	2, 600	1, 900	3, 450	2, 750	2, 350	1, 700	3, 100	2, 500	2, 150	1, 550
20	52	3, 800	2, 900	2, 200	1, 550	3, 250	2, 500	1, 900	1, 350	2, 950	2, 250	1, 700	1, 200	2, 650	2, 050	1, 550	1, 100
0	40	2, 950	2, 200	1, 550	950	2, 550	1, 900	1, 350	800	2, 250	1, 700	1, 200	750	2, 050	1, 550	1, 100	650

Table G.—Tolerable capacities of existing two-lane highways located in mountainous terrain, assuming an operating speed of 40–45 miles per hour and 5-percent truck traffic during 30th highest hour

Percentage of highway with passing sight distance of—		Average annual daily traffic volumes of two-lane highways with—															
		12-foot traffic lanes and passenger-car speeds at low volume of—				11-foot traffic lanes and passenger-car speeds at low volume of—				10-foot traffic lanes and passenger-car speeds at low volume of—				9-foot traffic lanes and passenger-car speeds at low volume of—			
1,500 feet	1,000 feet	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	60 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.
100	100	6, 550	6, 250	5, 800	5, 100	5, 650	5, 350	5, 000	4, 400	5, 050	4, 800	4, 450	3, 950	4, 600	4, 350	4, 050	3, 550
80	85	6, 000	5, 700	5, 200	4, 500	5, 150	4, 900	4, 450	3, 850	4, 600	4, 400	4, 000	3, 450	4, 200	4, 000	3, 650	3, 150
60	72	5, 300	5, 000	4, 450	3, 750	4, 550	4, 300	3, 850	3, 200	4, 100	3, 850	3, 450	2, 900	3, 700	3, 500	3, 100	2, 600
40	58	4, 600	4, 100	3, 600	2, 900	3, 950	3, 550	3, 100	2, 500	3, 550	3, 150	2, 750	2, 250	3, 200	2, 850	2, 500	2, 050
20	44	3, 850	3, 200	2, 500	1, 700	3, 300	2, 750	2, 150	1, 450	2, 950	2, 450	1, 900	1, 300	2, 700	2, 250	1, 750	1, 200
0	30	3, 050	2, 100	1, 500	950	2, 600	1, 800	1, 300	800	2, 350	1, 600	1, 150	750	2, 150	1, 450	1, 050	650

Table H.—Tolerable capacities of existing two-lane highways located in mountainous terrain, assuming an operating speed of 35–40 miles per hour and 5-percent truck traffic during 30th highest hour

Percentage of highway with passing sight distance of—		Average annual daily traffic volumes of two-lane highways with—															
		12-foot traffic lanes and passenger-car speeds at low volume of—				11-foot traffic lanes and passenger-car speeds at low volume of—				10-foot traffic lanes and passenger-car speeds at low volume of—				9-foot traffic lanes and passenger-car speeds at low volume of—			
1,500 feet	1,000 feet	55 M. p. h.	50 M. p. h.	45 M. p. h.	40 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	40 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	40 M. p. h.	55 M. p. h.	50 M. p. h.	45 M. p. h.	40 M. p. h.
100	100	7, 700	7, 450	7, 050	6, 150	6, 600	6, 400	6, 050	5, 300	5, 950	5, 750	5, 450	4, 750	5, 400	5, 200	4, 950	4, 300
80	83	7, 000	6, 800	6, 350	5, 550	6, 000	5, 850	5, 450	4, 750	5, 400	5, 250	4, 900	4, 250	4, 900	4, 750	4, 450	3, 900
60	67	6, 300	6, 200	5, 650	4, 650	5, 400	5, 350	4, 850	4, 000	4, 850	4, 750	4, 350	3, 600	4, 400	4, 350	3, 950	3, 250
40	51	5, 550	5, 400	4, 800	3, 650	4, 750	4, 650	4, 150	3, 150	4, 250	4, 150	3, 700	2, 800	3, 900	3, 800	3, 350	2, 550
20	36	4, 900	4, 550	3, 500	2, 400	4, 200	3, 900	3, 000	2, 050	3, 750	3, 500	2, 700	1, 850	3, 450	3, 200	2, 450	1, 700
0	20	4, 150	3, 000	1, 900	1, 150	3, 550	2, 600	1, 650	1, 000	3, 200	2, 300	1, 450	900	2, 900	2, 100	1, 350	800

The Economic Costs of Motor-Vehicle Accidents of Different Types

BY THE DIVISION OF HIGHWAY TRANSPORT RESEARCH
BUREAU OF PUBLIC ROADS

Reported [1] by ROBIE DUNMAN
Transportation Economist

This article discusses the frequencies and direct costs of different types of accidents involving passenger cars in Massachusetts during 1953.

Of the 131,500 accidents recorded in this study, nearly three-fourths were property-damage-only accidents, one-fourth were nonfatal-injury accidents, and less than half of one percent were fatal-injury accidents. For every dollar spent as a result of these accidents, nonfatal-injury accidents accounted for 57 cents; property-damage-only accidents, 40 cents; and fatal-injury accidents, 3 cents.

Collisions between passenger cars or passenger cars and other motor vehicles—by far the most frequent type of accident—accounted for 83 percent of the number and the same proportion of the cost of all accidents. Collisions with pedestrians, fixed objects, other objects, and noncollision types of accidents made up the remaining 17 percent.

Angle, rear-end, and head-on collisions represented nearly 81 percent of the number and 89 percent of the cost of all collisions between passenger cars or passenger cars and other motor vehicles. Angle collisions were the most frequent and were followed in order by rear-end and head-on collisions.

IN December 1947 the Highway Research Board recommended that the Bureau of Public Roads cooperate with the States in conducting studies of the economic costs of motor-vehicle accidents. These studies are now underway in Massachusetts, New Mexico, and Utah, and a fourth study is programed in Wisconsin. On the basis of preliminary discussion, it is anticipated that a fifth study will be started in Michigan during 1958.

This article reports data developed by the Massachusetts Department of Public Works and by the Massachusetts Registry of Motor Vehicles in cooperation with the Bureau of Public Roads. It is emphasized that the findings may not be typical or average for all States.

The number and direct cost of motor-vehicle traffic accidents involving passenger cars in Massachusetts during 1953 are shown by type of accident and severity of accident. Comparisons are made on the basis of cost per accident, per capita, per passenger car registered, per licensed operator, per mile of road, and per 100 million vehicle-miles of travel.

The purpose of this article is to present the cost of accidents in relation to the types of accidents in a way that will be helpful to groups and individuals who are trying to reduce traffic accidents and the resulting economic loss.

Statistical studies of the economic cost of motor-vehicle accidents are based on a probability sample of the accident experience of vehicle owners. They are designed to be accurate within 10 percent. By means of mailed questionnaires and through personal interviews with selected vehicle owners, their accident experience for one year is obtained. From these data the direct cost of accidents is estimated and correlated with the more important characteristics of accidents, including those peculiar to the highway and street facilities, the driver, and the vehicle. These studies are statewide and comprehensive. Because the data collected and analyzed in each State are so detailed and voluminous, this article is confined to one segment of the comprehensive study of traffic-accident costs in Massachusetts during 1953 and relates only to accidents in which passenger cars were involved. The accidents were motor-vehicle traffic accidents occurring on public roadways and involving motion.

Definitions

Direct costs are defined as the money value of damages and losses to persons and property that were the direct result of these accidents and which might have been saved had these accidents not occurred. Direct costs are composed of the money value of damage to property; hospitalization; services of physicians, dentists, and nurses; ambulance use; medicine; work time lost; damages awarded in excess of other direct costs; attorneys' services; court fees; and other miscellaneous but small items.

The type of collision was determined by the direction of travel of the vehicles involved before the collision, and not by what took place because of efforts on the part of drivers to avoid collision. Thus, any collision involving an intentional change of direction such as a right, left, or U-turn was classified as a turning movement, even though this may have resulted in a head-on or rear-end type of collision. Similarly, an angle collision resulted when two or more vehicles, each traveling in a straight line, came together at an intersection. Although one or both drivers may have swerved to avoid impact and collided in a sideswipe fashion, this was still classified as an angle collision. On the same basis, a collision involving a vehicle entering or leaving a parking space was classified as a parking maneuver even though the vehicle struck, or was struck by another, in a head-on, rear-end, or sideswipe fashion. A sideswipe occurred only when vehicles collided while overtaking and passing in the same direction, passing in the opposite direction, or passing a parked vehicle.

All other definitions are from the manual *Uniform Definitions of Motor-Vehicle Accidents 1947*, prepared under the auspices of the National Conference on Uniform Traffic Accident Statistics and published by the Federal Security Agency, U. S. Public Health Service.

In 1953 the population of Massachusetts was 4,773,000. There were 1,239,000 registered passenger cars and 1,858,000 licensed operators who drove their cars 11,628 million vehicle-miles over the Commonwealth's 24,500 miles of streets and highways. These passenger-car operators experienced 222,000 involvements in 131,500 accidents that resulted in a direct cost of $50,224,000.

Accident Severity

All motor-vehicle traffic accidents fall into one of three severity classes—property-damage-only accidents, nonfatal-injury accidents, and fatal-injury accidents.

Table 1 shows the accident experience of Massachusetts passenger-car operators during 1953 and brings into focus the numerical relation of accidents of different severity. The 33,270 nonfatal-injury accidents and the 315 fatal-injury accidents are in the ratio of 106: 1. This ratio is slightly more than 3 times the 35: 1 injury-to-fatal-accident ratio ordinarily used in estimating the cost of accidents. Whether or not this high ratio will hold in predominantly rural States will be known within a short time when results of the Utah and New Mexico studies are available.

It is apparent from table 1 that 3 out of 4 motor-vehicle traffic accidents resulted in property damage only, 1 out of 4 accidents resulted in a nonfatal injury, and only 1 in 417 accidents resulted in a fatal injury. It is also evident that there were 106 times as many nonfatal-injury accidents and 311 times as many property-damage-only accidents as there were fatal-injury accidents per 100 million vehicle-miles of travel.

On a population basis, there was 1 fatal

[1] This article was presented at the 37th Annual Meeting of the Highway Research Board, Washington, D. C., January 1958.

'able 1.—Motor-vehicle traffic accidents involving passenger cars in Massachusetts during 1953, classified by severity of accident

Number and rate of accidents	Severity of accident			All accidents
	Fatal injury	Nonfatal injury	Property damage only	
Number of accidents	315	33,270	97,915	131,500
Percent of total	0.2	25.3	74.5	100.0
Number of accidents per 100 million vehicle-miles of travel	[1] 3	286	842	1,131
Number of persons per accident	15,152	143	49	36
Number of registered passenger cars per accident	3,933	37	13	9
Number of licensed operators per accident	5,898	56	19	14

[1] Rounded from 2.7.

'able 2.—Direct cost of motor-vehicle traffic accidents involving passenger cars in Massachusetts during 1953, classified by severity of accident

Severity of accident	Total direct cost	Percent of total	Total direct cost—					
			Per accident	Per capita	Per passenger car registered	Per licensed operator	Per mile of road	Per 100 million vehicle-miles of travel
	1,000 dollars							1,000 dollars
Fatal injury	1,642	3.3	$5,213	$0.34	$1.32	$0.88	$67	14
Nonfatal injury	28,688	57.1	862	6.01	23.15	15.44	1,171	247
Property damage only	19,894	39.6	203	4.17	16.06	10.71	812	171
All accidents	50,224	100.0	382	10.52	40.53	27.03	2,050	432

ccident for every 15,152 persons, 1 nonfatal-ijury accident for every 143 persons, and 1 roperty-damage-only accident for every 49 ersons. The accident severity rate for pasenger cars registered was as follows: 1 fatal ccident for every 3,933 passenger cars, 1 nonatal-injury accident for every 37 passenger ars, and 1 property-damage-only accident or every 13 passenger cars. The accident everity rate for licensed passenger-car drivers vas 1 fatal-injury accident for every 5,898 rivers, 1 nonfatal-injury accident for every 6 drivers, and 1 property-damage-only accient for every 19 drivers.

The direct costs of motor-vehicle traffic ccidents in Massachusetts are presented in able 2. It should be remembered in considring these costs that they apply only to ccidents involving passenger cars, and they lo not include any indirect costs such as the resent value of future earnings and the overead cost of motor-vehicle accident insurance.

After comparing the number of accidents vith the costs shown in table 2, it is found that fatal accident with a direct cost of $5,213 is he equivalent of either 6 nonfatal-injury accilents with a direct cost of $862 each or 25 roperty-damage-only accidents with a direct ost of $203 each. However, it is also apparent that on the basis of 100 million vehicle-miles of travel, property-damage-only accidents cost 12 times as much and nonfatal-injury accidents, 18 times as much as the cost of fatal accidents.

Furthermore, table 2 shows that from the economic point of view, nonfatal-injury accidents were by far the most significant. They accounted for 57 cents of every accident direct-cost dollar. Property-damage-only accidents accounted for 40 cents of every accident direct-cost dollar, whereas the emotion-packed and often highly dramatized fatal accident accounted for only 3 cents. This comparison does not minimize the personal tragedy of fatal accidents.

The relation of direct cost of accidents to population, licensed operators, passenger cars registered, and road mileages are also given in table 2.

The per capita cost ranged from 34 cents for fatal-injury accidents to $6.01 for nonfatal-injury accidents, and the total direct cost for all accidents averaged $10.52 per capita.

On the basis of direct cost per passenger-car registered, the range was from $1.32 for fatal-injury accidents to $23.15 for nonfatal-injury accidents. The total direct cost of all accidents was $40.53 per passenger car registered.

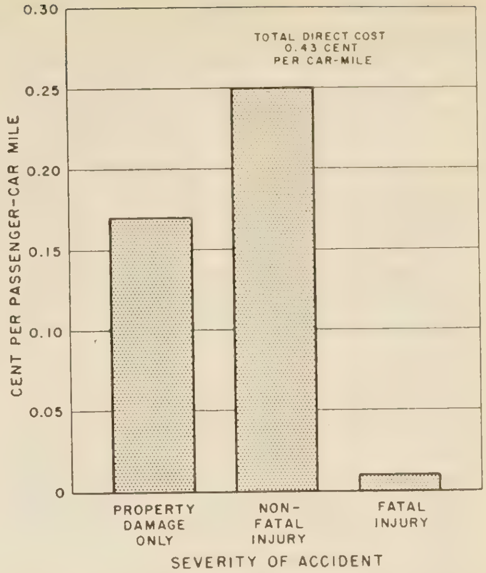

Figure 1.—Direct cost of traffic accidents per passenger-car mile of travel in Massachusetts during 1953, classified by severity of accident.

Similar comparisons of costs per licensed passenger-car operator and per mile of road indicated a range of 88 cents to $15.44 and $67 to $1,171 for fatal- and nonfatal-injury accidents, respectively.

Figure 1 shows that the direct cost of operating a passenger car 1 mile was 0.17 cent for property-damage-only accidents, 0.25 cent for nonfatal-injury accidents, and 0.01 cent for fatal-injury accidents, or a total of 0.43 cent for each mile of passenger-car operation.

Type of Accidents

Motor-vehicle traffic accidents fall into one of five types: collision between motor vehicles, collision with pedestrians, collision with fixed objects, collision with other objects, and the noncollision-type accidents in which the vehicle turns over in the road or runs off the road.

It is evident in table 3 that of all accident types, collisions between motor vehicles were by far the most numerous. More than 8 out of 10 accidents involving passenger cars were of this type. Furthermore, out of a total of 1,131 accidents per 100 million vehicle-miles of travel, 943 involved passenger-car collisions with other motor vehicles.

Second from the standpoint of numbers of accidents was passenger-car collisions with objects, fixed and otherwise. This type accounted for 1 out of 10 accidents. About 1 in every 20 accidents involved passenger cars and pedestrians, and less than 1 in 50 accidents were the noncollision type. However, the number of accidents alone is not a measure of the relative economic significance of accidents of different types; both the number and the severity of accidents must be considered.

By relating the number of accidents as shown in table 3 with population, passenger cars registered, and licensed drivers, accident rates were as follows:

Table 3.—Motor-vehicle traffic accidents involving passenger cars in Massachusetts during 1953, classified by type of accident

Number and rate of accidents	Passenger cars colliding with—				Non-collision accidents	All accidents
	Other motor vehicles	Pedestrians	Fixed objects	Other objects		
Number of accidents	109,700	5,900	7,300	6,400	2,200	131,500
Percent of total	83.4	4.5	5.5	4.9	1.7	100.0
Number of accidents per 100 million vehicle-miles of travel	943	51	63	55	19	1,131
Number of persons per accident	44	809	654	746	2,170	36
Number of passenger cars registered per accident	11	210	170	194	563	9
Number of licensed operators per accident	17	315	255	290	845	14

Table 4.—Direct cost of motor-vehicle traffic accidents involving passenger cars in Massachusetts during 1953, classified by type of accident

Type of accident	Total direct cost	Percent of total	Total direct cost— Per accident	Per capita	Per passenger car registered	Per licensed operator	Per mile of road	Per 100 million vehicle-miles of travel
	1,000 dollars							1,000 dollars
Passenger-car collision with—								
Other motor vehicles	41,816	83.3	$381	$8.76	$33.75	$22.50	$1,707	360
Pedestrians	3,375	6.7	572	.71	2.72	1.82	138	29
Fixed objects	3,023	6.0	414	.63	2.44	1.63	123	26
Other objects	673	1.3	105	.14	.54	.36	27	6
Noncollision accidents	1,337	2.7	608	.28	1.08	.72	55	11
All accidents	50,224	100.0	382	10.52	40.53	27.03	2,050	432

There was 1 collision between motor vehicles for every 44 persons, 1 collision with pedestrians for every 809 persons, 1 collision with fixed objects for every 654 persons, 1 collision with other objects for every 746 persons, and 1 noncollision accident for every 2,170 persons.

On the basis of passenger cars registered, there was 1 collision between motor vehicles for every 11 passenger cars, 1 collision with pedestrians for every 210 passenger cars, 1 collision with fixed objects for every 170 passenger cars, 1 collision with other objects for every 194 passenger cars, and 1 noncollision accident for every 563 passenger cars.

Considering the number of licensed operators, there was 1 collision between motor vehicles for every 17 drivers, 1 collision with pedestrians for every 315 drivers, 1 collision with fixed objects for every 255 drivers, 1 collision with other objects for every 290 drivers, and 1 noncollision accident for every 845 drivers.

The average direct cost for each type of accident, as shown in table 4, was found to be as follows: Collisions between motor vehicles, $381; collisions of passenger cars with pedestrians, $572; collisions with fixed objects, $414; collisions with other objects, $105; and noncollision accidents, $608.

These average costs reflect the severity of accidents of different types rather than their economic importance. It was found that noncollision accidents were the most severe type of accident, and following in order were passenger-car collisions wit pedestrians, passenger-car collisions wit fixed objects, collisions between motor ve hicles, and passenger-car collisions with othe objects.

The overriding economic impact of colli sions between motor vehicles is made quit clear in table 4. Out of every accident direct cost dollar, 83 cents applied to this type c accident. On the basis of 100 million vehicle miles of travel, collisions between moto vehicles cost five times as much as all othe types of accidents combined.

Other types of accidents ranked in orde of cost (fractional parts of a dollar) were a follows: passenger cars colliding with pedes trians, 7 cents; collisions with fixed objects 6 cents; noncollision accidents, 3 cents; an collisions with other than fixed objects, 1 cent

By relating the direct cost data in table 4 t population, passenger cars registered, license operators, and road mileages, accident cos rates were determined as follows:

The per capita direct cost was $8.76 fo collisions with other motor vehicles, 71 cent for collisions with pedestrians, 63 cents fo collisions with fixed objects, 14 cents fo collisions with other objects, and 28 cents fo noncollision accidents.

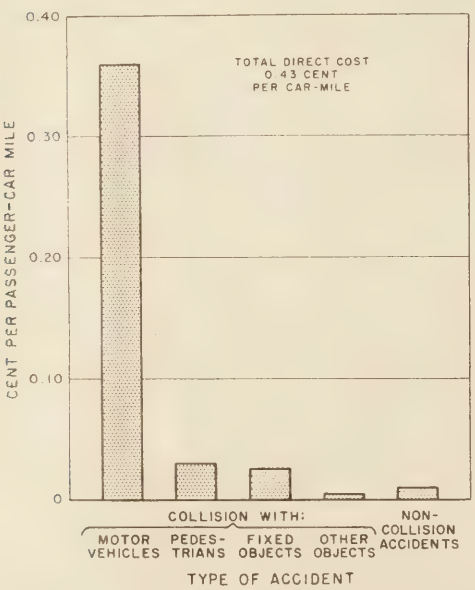

Figure 2.—Direct cost of traffic accidents per passenger-car mile of travel in Massachusetts during 1953, classified by type of accident.

Table 5.—Number of collisions between passenger cars or passenger cars and other moto vehicles in Massachusetts during 1953, classified by type of collision

Number and rate of collisions	Type of collision— Angle	Rear-end	Head-on	Sideswipe, same direction	Parking maneuver	Turning movement	Backing in traffic lane	Sideswipe, opposite direction	All collisions
Number of collisions	53,200	22,500	12,800	7,100	5,200	4,800	2,600	1,500	109,700
Percent of total	48.5	20.5	11.7	6.5	4.7	4.4	2.4	1.3	100.0
Number of collisions per 100 million vehicle-miles of travel	458	193	110	61	45	41	22	13	943
Number of persons per collision	90	212	373	[1] 225	------	------	------	------	44
Number of passenger cars registered per collision	23	55	97	[1] 58	------	------	------	------	11
Number of licensed operators per collision	35	83	145	[1] 88	------	------	------	------	17

[1] Includes "sideswipe, same direction" and the four remaining types of collisions.

Table 6.—Direct cost of collisions between passenger cars or passenger cars and othe motor vehicles in Massachusetts during 1953, classified by type of collision

Type of collision	Total direct cost	Percent of total	Total direct cost— Per collision	Per 100 million vehicle-miles of travel
	1,000 dollars			1,000 dollars
Angle	17,386	41.6	$327	150
Rear-end	10,842	25.9	482	93
Head-on	9,078	21.7	709	78
Sideswipe, same direction	1,958	4.7	276	17
Parking maneuver	599	1.4	115	5
Turning movement	1,114	2.7	232	10
Backing in traffic lane	133	.3	51	1
Sideswipe, opposite direction	706	1.7	471	6
All collisions	41,816	100.0	381	360

'able 7.—Number of accidents involving passenger cars (other than collisions between motor vehicles) in Massachusetts during 1953, classified by type of accident

Number and rate of accidents	Passenger cars colliding with—			Noncollision accidents	Total
	Pedestrians	Fixed objects	Other objects		
Number of accidents	5,900	7,300	6,400	2,200	21,800
Percent of total	27.1	33.5	29.3	10.1	100.0
Number of accidents per 100 million vehicle-miles of travel	51	63	55	19	188
Number of persons per accident	809	654	746	2,170	219
Number of registered passenger cars per accident	210	170	194	563	57
Number of licensed operators per accident	315	255	290	845	85

able 8.—Direct cost of accidents involving passenger cars (other than collisions between motor vehicles) in Massachusetts during 1953, classified by type of accident

Type of accident	Total direct cost	Percent of total	Total direct cost—				
			Per accident	Per capita	Per passenger car registered	Per licensed operator	Per 100 million vehicle-miles of travel
	1,000 dollars						*1,000 dollars*
Passenger-car collision with—							
Pedestrians	3,375	40.1	$572	$0.71	$2.72	$1.82	29
Fixed objects	3,023	36.0	414	.63	2.44	1.63	26
Other objects	673	8.0	105	.14	.54	.36	6
Noncollision accidents	1,337	15.9	608	.28	1.08	.72	11
All accidents	8,408	100.0	386	1.76	6.78	4.53	72

The direct cost per passenger-car registered 'as $33.75 for passenger-car collisions with her motor vehicles, $2.72 for collisions with :destrians, $2.44 for collisions with fixed ɔjects, 54 cents for collisions with other ɔjects, and $1.08 for noncollision accidents.

The direct cost per licensed passenger-car perator was $22.50 for passenger-car colli-ɔns with other motor vehicles, $1.82 for ɪllisions with pedestrians, $1.63 for collisions ith fixed objects, 36 cents for collisions with her objects, and 72 cents for noncollision :cidents.

The direct cost per mile of road was $1,707 r passenger-car collisions with other motor :hicles, $138 for collisions with pedestrians, 23 for collisions with fixed objects, $27 for ɪllisions with other objects, and $55 for ɔncollision accidents.

Figure 2 shows that the cost of operating a ɪssenger car 1 mile was 0.36 cent for collisions :tween motor vehicles, 0.029 cent for pas-nger-car collisions with pedestrians, 0.026 nt for passenger-car collisions with fixed ɔjects, 0.006 cent for passenger-car collisions ith other objects, 0.011 cent for noncollision :cidents, and a total accident direct cost of 43 cent per mile.

Collisions Between Motor Vehicles

All collisions between motor vehicles fall to one of the eight collision types as shown table 5. The different types of collisions e listed in the order of their numerical ɪportance. Angle collisions, which were by r the most numerous, accounted for nearly half of the collisions between motor vehicles. About 1 out of 5 collisions was a rear-end collision, and one in 9 was a head-on collision. These three collision types accounted for 80 percent of all collisions between motor vehicles.

There were less than half as many rear-end collisions and approximately one-fourth as many head-on collisions as there were angle collisions per 100 million vehicle-miles of travel. On this same basis there were more rear-end collisions than there were collisions of the following five types combined: sideswipes in the same direction, parking-maneuver collisions, turning-movement collisions, backing-in-traffic-lane collisions, and sideswipes in the opposite direction. In fact, these five types together accounted for only one-fifth of all collisions between motor vehicles.

Collision rates based on travel, population, registered vehicles, and licensed operators further emphasize the high frequencies of the angle type of collision.

It is evident from table 6 that angle collisions ranked far above all other types in economic importance. Almost 42 cents of the direct-cost dollar spent for collisions between motor vehicles resulted from angle collisions. The rear-end collision ranked second and accounted for almost 26 cents of every dollar spent for collision accidents. Following closely were head-on collisions, which accounted for almost 22 cents of every collision-cost dollar. The five remaining types of collisions were of considerably less economic importance. Together they accounted for less than 11 cents of every dollar.

A comparison of the average direct cost per collision reveals that the head-on type of collision was by far the most expensive of all types, and was followed in order by rear-end, sideswipe (opposite direction), and angle collisions.

The direct cost of operating a passenger car 1 mile, as shown in figure 3, was 0.15 cent for angle collisions, 0.09 cent for rear-end collisions, 0.08 cent for head-on collisions, and 0.04 cent for all other types of collisions. The total direct cost of all types of collisions between motor vehicles was 0.36 cent per mile of passenger-car operation.

Accidents, Excluding Collisions Between Motor Vehicles

Passenger-car accidents other than collisions between motor vehicles are ordinarily classified into eight types as follows: collisions with fixed objects, collisions with pedestrians, collisions with bicycles, collisions with animals or animal-drawn vehicles, collisions with railroad trains, collisions with streetcars, collisions with other objects, and noncollision accidents. However, since accidents involving collisions with bicycles, animals or animal-drawn vehicles, trains, and streetcars together accounted for less than 3,000 of the almost 22,000 passenger-car accidents, excluding collisions between motor vehicles, these four accident types were combined with other objects in tables 7 and 8.

Of the 21,800 accidents other than collisions between motor vehicles, 1 out of 10 was a

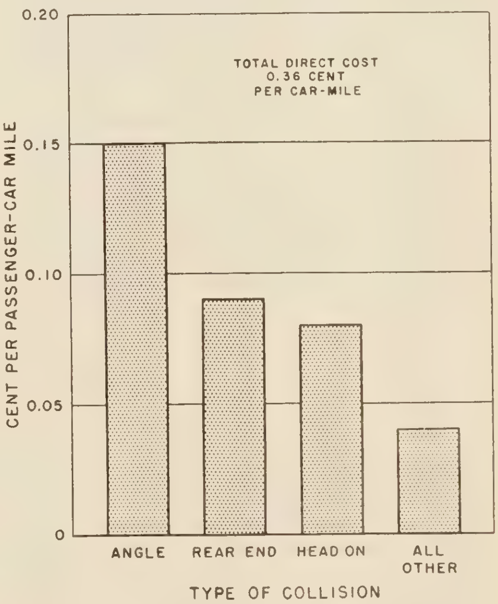

Figure 3.—Direct cost of collisions between passenger cars or passenger cars and other motor vehicles per passenger-car mile of travel in Massachusetts during 1953, classified by type of collision.

noncollision type in which the vehicle turned over in the road or ran off the road without striking anything. In 9 out of 10 accidents, the passenger car struck something other than another motor vehicle.

The costs shown in table 8 represent 16.6 percent of the total direct cost of all motor-vehicle traffic accidents experienced by Massachusetts passenger-car drivers during 1953. Not included in the table are collisions between motor vehicles.

Among the four types of accidents shown, collisions with pedestrians were of the greatest economic importance. Collisions with fixed objects ranked second, noncollision accidents ranked third, and collisions with other objects ranked last. On the basis of cost per accident, the noncollision-type accident ranked first and was followed by collisions with pedestrians, collisions with fixed objects, and collisions with other objects.

Direct Costs Summarized

The severity-class dollar illustrated in figure 4 is representative of the $50,224,000 spent on accidents in Massachusetts during 1953. This diagram shows that the cost of nonfatal-injury accidents is greater than that of fatal accidents and property-damage-only accidents combined. The diagram also portrays the minor economic role of fatal accidents.

The accident-type dollar, as diagramed in figure 4, brings accidents of different types into proper economic perspective. It illustrates the overriding economic importance of collisions between motor vehicles.

Figure 4 also illustrates the allocation of costs for the various types of collisions between motor vehicles. Angle, rear-end, and head-on collisions accounted for 90 cents of the collision-between-motor-vehicles dollar.

The noncollision and collision-with-objects dollar, as diagramed in figure 4, is the equivalent of a 17-cent segment of the accident-type dollar. It is representative of the $8,408,000 spent for noncollision accidents and passenger-car collisions with objects other than another motor vehicle.

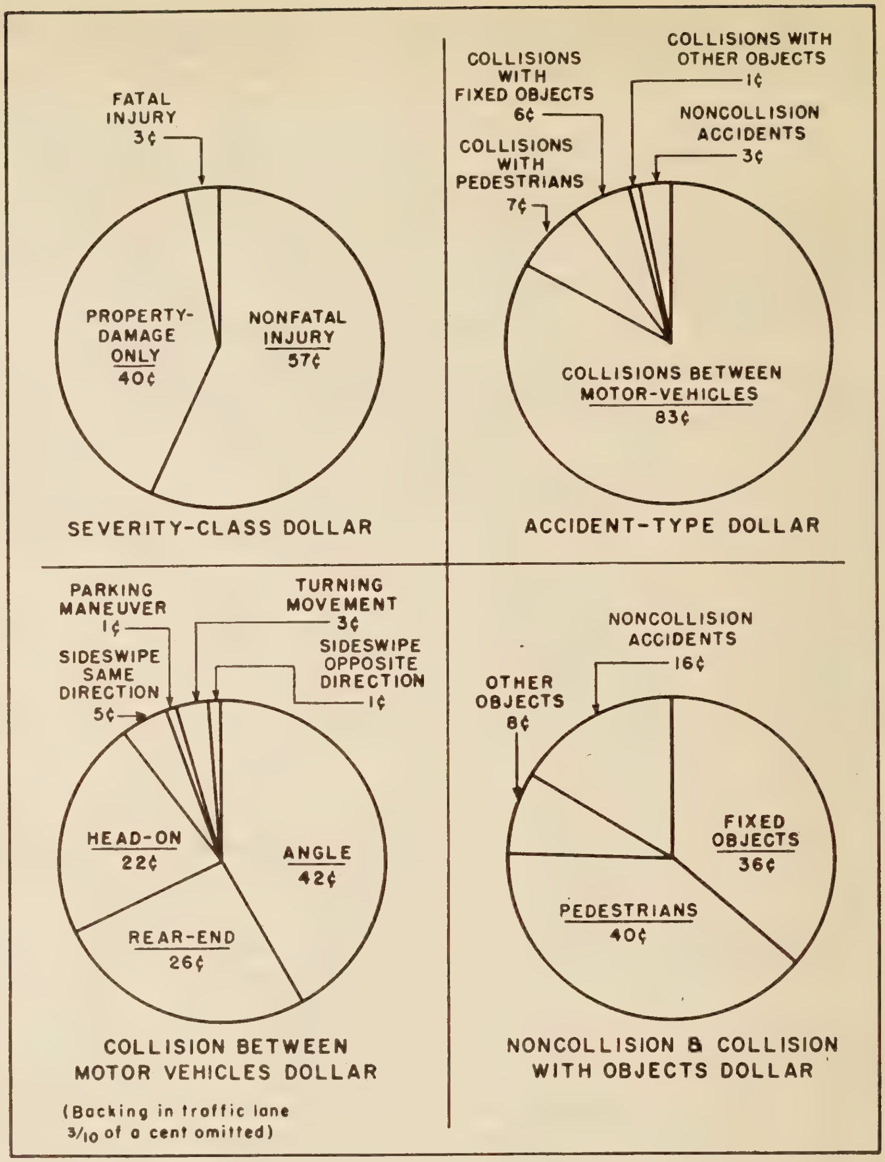

Figure 4.—Direct cost of passenger-car accidents shown as fractional parts of a dollar and classified according to severity of accident, type of accident, type of motor-vehicle collision, and other types of collision and noncollision accidents.

Continued from page 37)

weighted average truck speed was entered in figure 6 to determine from the curve the truck equivalent in terms of passenger cars. Capacity analysis then was completed as previously described.

Descriptions of working procedures and the application of these data in estimating the requirements for truck lanes or other design modifications are discussed also in the manuals (*7, 8*) published by the State highway departments of Kentucky and Tennessee.

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Jse of the Swiss Hammer for Estimating the Compressive Strength of Hardened Concrete

Y THE DIVISION OF PHYSICAL RESEARCH
UREAU OF PUBLIC ROADS

Reported [1] by WILLIAM E. GRIEB, Highway Physical Research Engineer

A simple and portable instrument for use estimating the compressive strength of ırdened concrete in place has been de- loped recently. The device, popularly ıown as the Swiss Hammer, is designed for eld use and is not intended as a substitute r control testing. It is being used in the eld to gage increases in concrete strength ith age and in locating low-strength areas hen laboratory tests of control concrete linders or other conditions indicate that ıch areas might exist. It is also useful in ırveys of old structures. The results of the sts given in this article show that factors ıch as surface smoothness, surface mois- ıre condition, and type of coarse aggregate fect the strength values obtained by the e of the device.

A TEST METHOD which is simple, quick, and nondestructive has been developed Ernest Schmidt, a Swiss engineer, for esti- ating the compressive strength of hardened ncrete in place. The device consists of a eel plunger or hammer, free to travel in a ıbular frame. When the head of the hammer pressed against the surface of concrete, the ımmer is retracted into the frame against e force of a tension spring. When the head completely retracted, the spring is auto- atically released driving the hammer against e concrete. A small sliding pointer indicates e rebound of the hammer on a graduated ale. The scale is 75 mm. in length, and ads from zero to 100 in equally spaced divi- ons. The amount of this rebound, *R*, was und by the inventor to be related to the mpressive strength of concrete.

[1] This article was presented at the 37th Annual Meeting the Highway Research Board, Washington, D. C., Jan- ry 1958.

A number of research organizations have studied the performance of the Swiss Hammer both in the laboratory and in the field. The consensus of their reports is that the empirical relationships between hammer rebound and strength are affected by moisture conditions of the concrete and type of aggregate, thus limiting the usefulness of the hammer to cases where an approximation of strength is all that is required. However, these reports do not include sufficient data to determine fully the capabilities of this instrument.

Testing Procedure

The surface of the concrete selected for test should be smooth and free from any rough spots or honeycomb. A surface produced by form work or troweling is usually satisfactory. When necessary, a smooth surface may be prepared by rubbing with a carborundum stone an area approximately 2 inches in diameter. A suitable stone is furnished in the carrying case of the apparatus.

In performing the test, the hammer is held perpendicular to the surface of the concrete and pressed against it until the hammer is released and strikes the surface of the concrete. While the device is still pressed firmly against the concrete, a button on the side of the instrument is pressed which locks the pointer in position. This permits the removal of the device to facilitate reading the amount of rebound. The apparatus is shown in figure 1.

For any selected area, five or more rebound readings are taken and the average of these readings is used to estimate the compressive strength. Areas where the reinforcing steel is known to be close to the surface, or where the coarse aggregate is exposed, are avoided.

The manufacturer of the instrument furnished a graph showing the relation between the compressive strength of the concrete and the rebound readings. This graph has been reproduced as figure 2. The data for establishing the relation represented by the curve were based on tests by the Swiss Federal Testing Laboratory. The curve for estimating the compressive strength shows values of rebound obtained when the hammer is held in a horizontal position against a vertical concrete surface. For other than horizontal positions of the hammer, a correction factor should be applied to the rebound readings before using the curve for estimating the strength of the concrete. A chart giving these correction factors was furnished by the manufacturer. These factors vary with the angle from the horizontal and the amount of the rebound; as the rebound reading increases, the correction factor decreases. For example with a rebound reading of 30, the corrections applied are as follows:

Angle from horizontal	*Correction factor*
(upward)	
90°	−6
60°	−5
30°	−3
0	0
(downward)	
30°	+2
60°	+3
90°	+4

Laboratory Tests

To determine the value of the Swiss Hammer as a tool for use in estimating the strength of concrete used in highway construction, three series of laboratory tests were made as well as numerous associated studies.

Series 1

The specimens used in this series were 6- by 12-inch cylinders submitted from various field projects. The concretes covered a wide variation in mixes and materials. All tests were made on specimens in a moist condition. Rebound readings were taken on the sides of the cylinders just prior to tests for compressive strength. The cylinders were tested in a vertical position with the side of the cylinder resting against an 8- by 12-inch machined-steel plate which in turn was supported by a wall of the laboratory. The hammer was held horizontal and perpendicular to the side

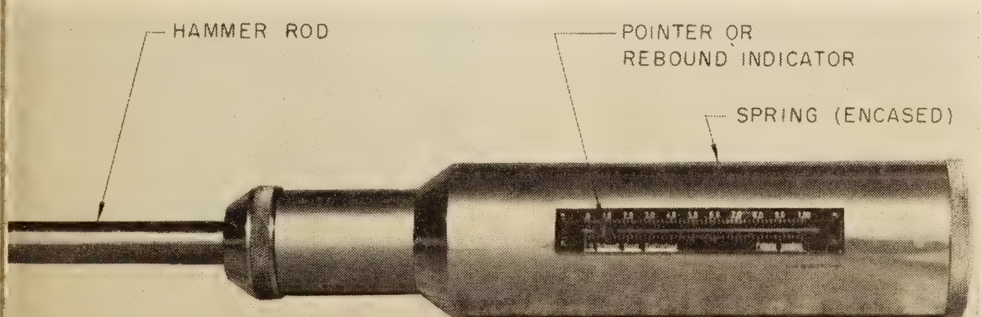

Figure 1.—Swiss rebound hammer.

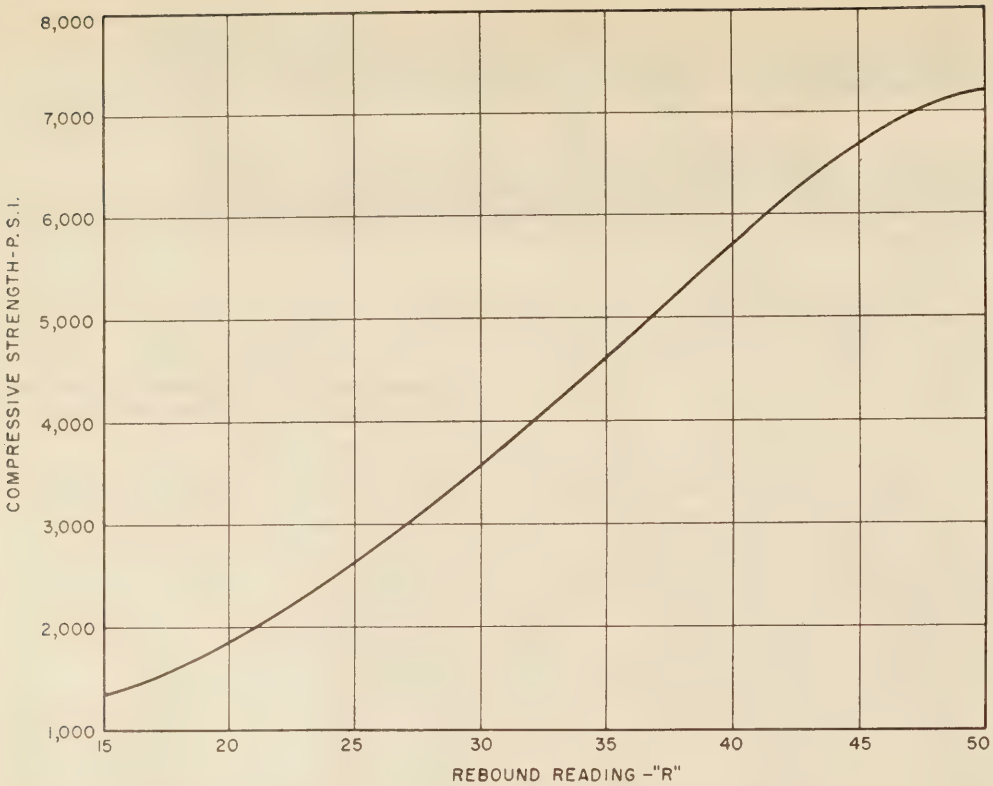

Figure 2.—Relation between compressive strengths and rebound readings as determined by the manufacturer.

of the cylinder. Usually 12 readings were taken on the side of each cylinder—3 readings on each quadrant with one reading 1 inch from the top, one at the center, and the other 1 inch from the bottom. Immediately after the rebound readings were taken, the cylinders were tested for compressive strength in a 400,000-pound hydraulic testing machine.

The results of the impact hammer and compressive strength tests on these cylinders are shown in figure 3. The upper curve represents the average relation between the rebound readings and the actual compressive strength. The strengths as shown by this curve are approximately 50 percent higher than the compressive strengths corresponding to the same rebound readings as shown in the curve furnished with the hammer. For example, the compressive strength for a rebound reading of 20, as determined from the curve for this series of tests, would be 2,750 p.s.i. as compared with 1,850 p.s.i. from the manufacturer's curve. For a rebound reading of 30, the compressive strength from the series 1 tests would be 5,300 p.s.i. as compared with only 3,600 p.s.i. from the manufacturer's curve for the same rebound reading.

The results of these tests indicated that the concrete cylinders held in the manner described did not have enough mass or rigidity to give reliable rebound readings, and that some of the energy from the blow may have been absorbed by movement of the cylinders.

Series 2

A second series of tests was made on another group of 6- by 12-inch cylinders submitted from projects under construction. To hold the cylinder firmly while the readings were taken with the hammer, each cylinder was put in the compression testing machine and a small load applied. A load of approximately 300 p.s.i. was found sufficient. Tests showed that greater loads had no effect on the rebound readings. After the rebound readings were taken, the cylinders were tested for compressive strength.

The results of these tests are shown in figure 4. The compressive strength for any rebound reading, as determined from the upper curve in figure 4, is approximately 12 percent higher than the compressive strength based on the curve submitted by the manufacturer of the hammer. Figure 5 shows the Swiss Hammer being used in the series 2 laboratory tests.

Series 3

In series 3 tests, the effect of type of coarse aggregate on the rebound-compressive strength relation was studied. All of the concrete cylinders were made in the laboratory and were tested as described in series 2. In the first part of the series (series 3A), four different gravels were used in making the cylinders tested. The results of the tests on these specimens are shown in figure 6. The spread in compressive strength among the curves representing the concrete prepared with the four gravel coarse aggregates varied from 250 to 600 p.s.i.

In the second part of this series, comparisons were made between concrete prepared with a siliceous gravel and crushed limestone. The curves giving the average relation between rebound readings and compressive strength for concrete containing these aggregates are shown in figure 7. The curve for the concrete prepared with crushed stone aggregate indicated about 25 percent greater strength for a given rebound reading than for the concrete prepared with gravel.

The curve for the relation between rebound readings and compressive strength for the gravel concrete corresponds very closely to that furnished by the manufacturer and shown in figure 2.

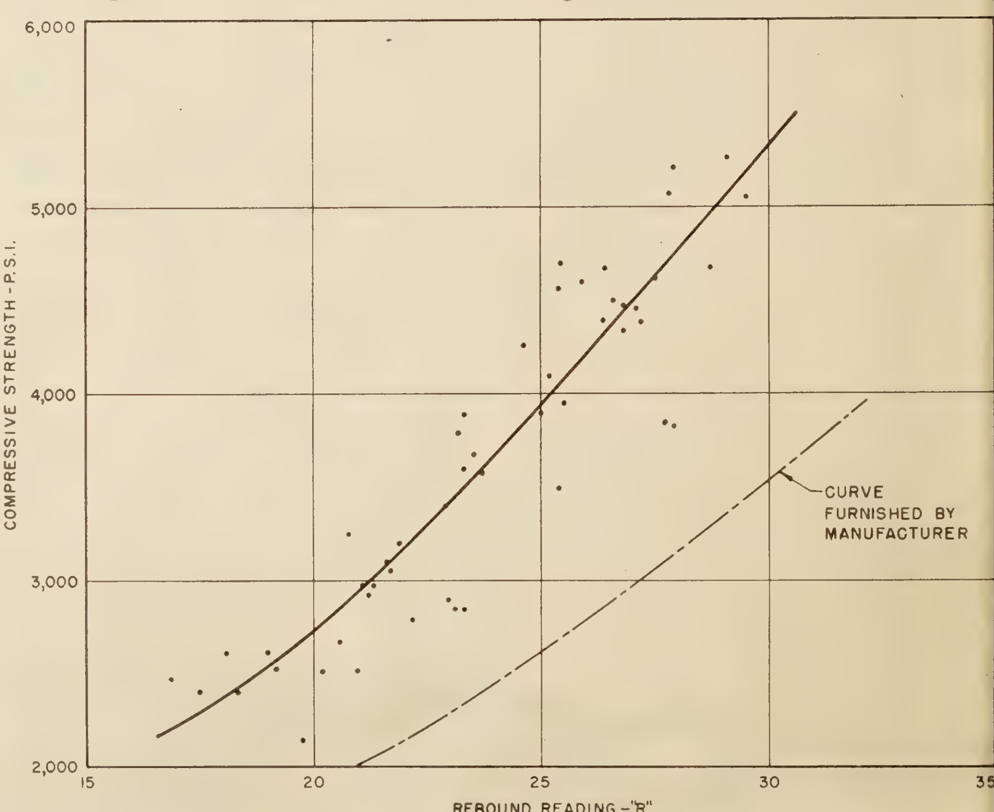

Figure 3.—Relation between compressive strengths and rebound readings on 6- by 12-inch concrete cylinders—series 1 tests.

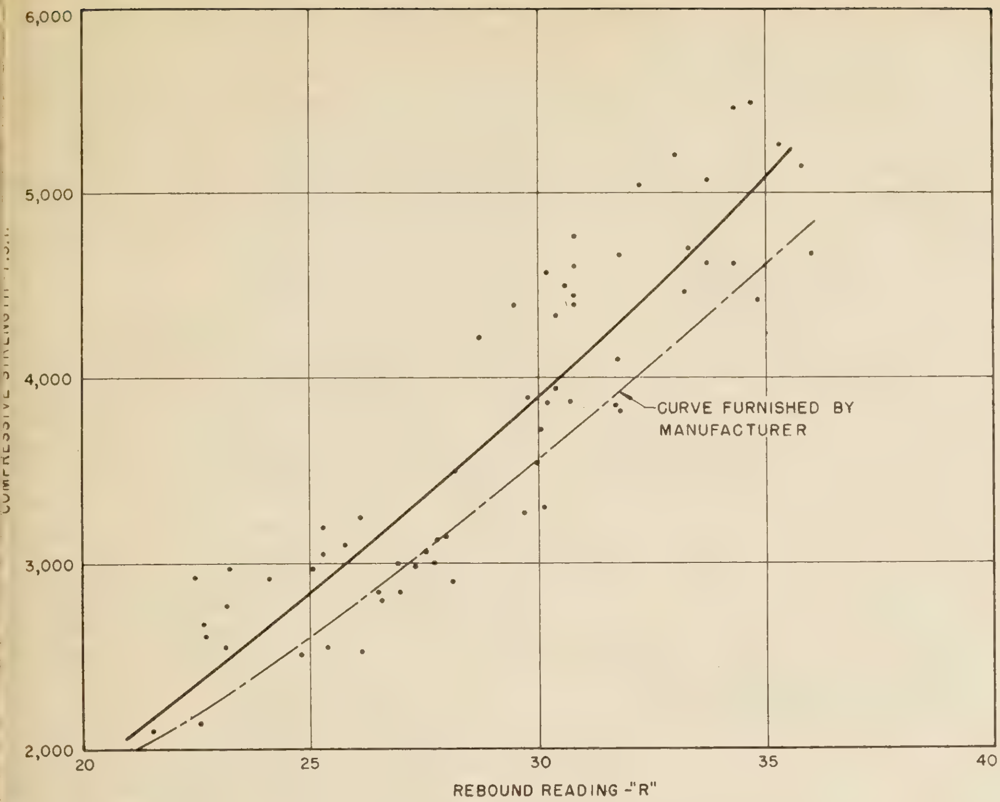

Figure 4.—Relation between compressive strengths and rebound readings on 6- by 12-inch concrete cylinders—series 2 tests.

These tests show that type of coarse aggregate is a governing factor in the rebound-compressive strength relation. This means that the Swiss Hammer is of most value in making comparative tests on concrete prepared with the same coarse aggregate. If comparisons between concretes prepared with different aggregates are desired, curves for the rebound-compressive strength relation for each aggregate should be obtained.

Associated Tests

Rebound readings were taken on the top and bottom of cylinders as cast prior to capping as well as on the sides. The readings were taken as described in the series 1 tests. There was considerable difference in the readings on the top, bottom, and sides of the same cylinder. The results are shown in table 1. This table also shows the estimated compressive strengths, which correspond to these readings, taken from the curve furnished by the manufacturer and the actual compressive strength of the concrete.

The average of the readings taken on the bottoms of all of the cylinders was 23 percent higher than the average of the readings taken on the sides of the cylinders, whereas the average of the readings on the top was only 5 percent higher. An explanation for some of this difference could be a difference between the quality of the concrete in the top and bottom of the cylinder. It is also possible that the cylinders were in a more rigid position when the readings were taken on the top and bottom than they were when readings were taken on the side.

Rebound readings were taken on a few cylinders in a dry condition and then on the same cylinders after immersion in water for 24 hours. The readings on the cylinders in a dry condition in all cases were larger than those in a moist condition. The results of these tests are shown in table 2. The estimated compressive strengths taken from the manufacturer's curve are also shown in this table.

A study was made to determine whether the rebound readings increased with age as the compressive strength increased. Of 12 concrete cylinders made from the same batch of concrete, 4 were tested at an age of 5 days, 4 at 10 days, and 4 at 20 days. The results of these tests are shown in table 3. The estimated compressive strength and the actual compressive strength of this concrete are also shown. The increase in rebound values was approximately proportional to the increase in the actual compressive strength.

Studies were made of the uniformity of the concrete in 6- by 6- by 21-inch beams, and

Figure 5.—The Swiss Hammer as used in the series 2 tests.

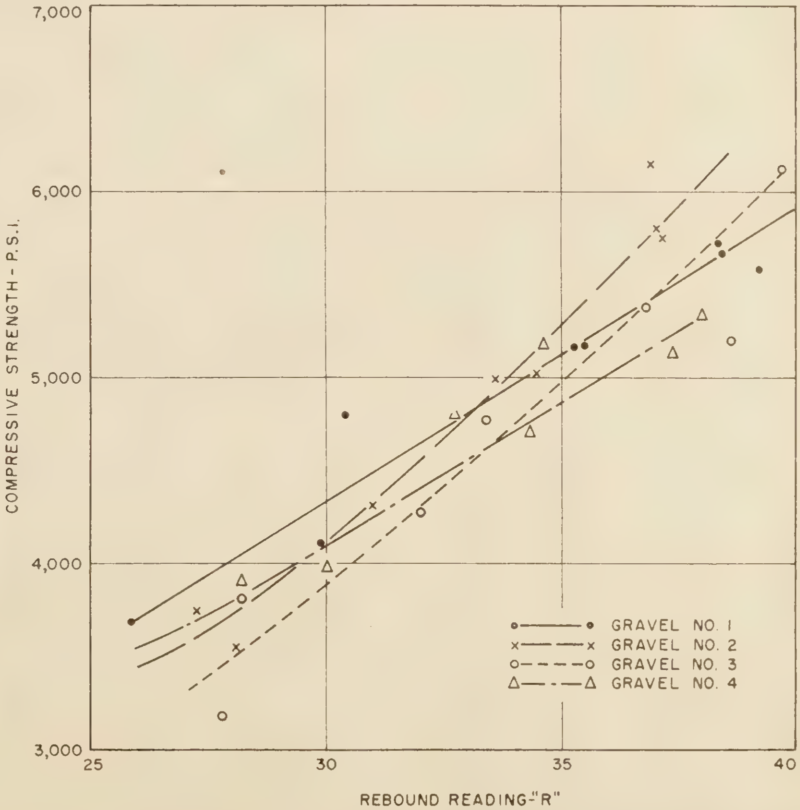

Figure 6.—Effect of gravel from different sources on rebound readings of 6- by 12-inch concrete cylinders—series 3A tests.

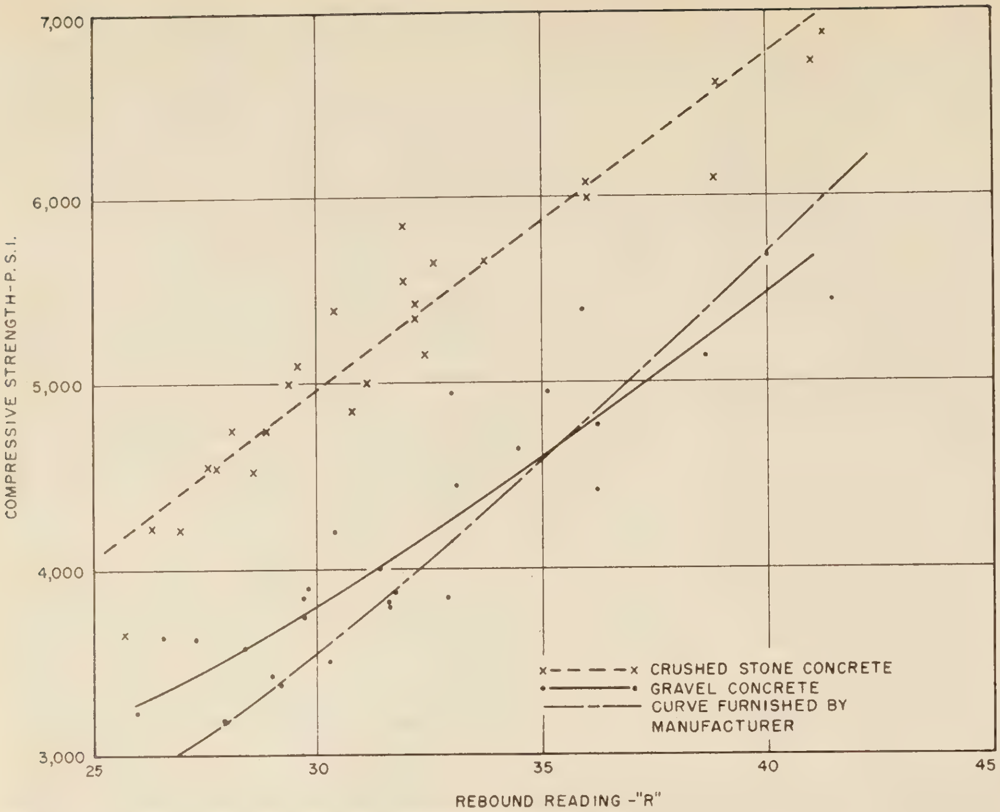

Figure 7.—Effect of type of coarse aggregate on rebound readings of 6- by 12-inch concrete cylinders—series 3B tests.

rebound readings were made on the sides, top, bottom, and ends of 29 beams. Five tests were made on the ends of each beam and 10 tests on each of the other faces. Average values for the entire group of beams were as follows:

Face of beam	*Rebound value*
Side	25. 5
Top	23. 6
Bottom	26. 1
End	28. 2

It is believed that these values correctly reflect slight differences between the quality of the concrete in different faces of the beams. With consideration given to the tendency of concrete to "bleed," the bottom of a beam should be more dense and have a higher rebound reading than the sides or the top. The rebound tests at the ends of the beams were made on a concrete specimen with a depth of 21 inches—3½ times the depth of concrete at any other point. This may be the reason for the greater readings.

A study was also made of the relation between rebound readings taken on the ends of 6- by 6- by 21-inch beams and readings taken on the sides of 6- by 12-inch cylinders. A beam and a corresponding cylinder were made from the same batch of concrete. Half of the total number of specimens contained gravel coarse aggregate and the other half contained crushed stone. The readings were taken on the beams held against the wall and the cylinders were placed in the testing machine with a small applied load as described in the series 2 tests. The beams were tested for flexural strength after the rebound readings were made.

The rebound values are shown in table 4 for both beams and cylinders, together with the actual compressive strengths of the cylinders and the flexural strengths of the beams. The average of all rebound readings on the cylinders for gravel concrete was 31.2 as compared with 30.9 for the beams made with gravel concrete. The average rebound reading was 32.3 for both cylinders and beams made from stone concrete.

There appears to be a definite relation between rebound readings taken on the ends of the beams and the flexural strength for this series of tests. This relation is shown in figure 8.

The Swiss Hammer could be used to estimate the flexural strength of paving concrete. Readings taken on control beams would indicate increases in flexural strength with age. From these readings, the age at which flexural strength tests should be made to meet specification requirements may be determined. This would reduce the number of control beams necessary.

Field Tests

The Swiss Hammer was used to estimate the strength of several concrete structures in the field. In one case, tests were made on three beam sections cast for post tensioning for use in a concrete bridge. Readings were taken on all beams prior to stressing and on one beam after stressing.

The concrete used in the beams was made with gravel aggregate. Each beam measured approximately 3 feet by 3 feet by 75 feet. Rebound readings were taken along the length of the beam at intervals of about 3 feet from one end to the center. Three readings were taken at each location: one reading 5 to 10 inches from the top, one at the center, and the other about 10 inches from the bottom. The beams were cured with wet burlap on the job site and were in a moist condition when readings were taken. Figure 9 shows the hammer being used on these beams.

Concrete cylinders for control were cast at the same time the beams were made. These were stored on the job for 5 days and then taken to a laboratory for moist storage and testing.

The rebound readings on the beams, the estimated compressive strength of the concrete in each beam as obtained from the curve furnished by the manufacturer (fig. 2), and the actual strengths of the test cylinders are shown in table 5. The average estimated compressive strengths of the beams were approximately 20 percent greater than the average compressive strengths of the test cylinders. Differences in curing and testing

Table 1.—Rebound readings on top, bottom, and side of 6- by 12-inch concrete cylinders

Cylinder number	Average rebound reading on side [1]	Estimated compressive strength [2]	Average rebound reading on top [1]	Estimated compressive strength [2]	Average rebound reading on bottom [1]	Estimated compressive strength [2]	Actual compressive strength [3]
		P. s. i.		*P. s. i.*		*P. s. i.*	*P. s. i.*
1	18. 3	1, 640	20. 8	1, 960	24. 2	2, 480	2, 410
2	19. 2	1, 750	22. 2	2, 160	23. 8	2, 420	3, 210
3	20. 8	1, 960	21. 8	2, 100	27. 4	3, 050	3, 250
4	21. 3	2, 040	21. 6	2, 050	29. 4	3, 420	2, 980
5	21. 3	2, 040	22. 0	2, 130	25. 4	2, 690	3, 290
6	22. 2	2, 160	22. 0	2, 130	29. 2	3, 380	2, 800
7	22. 8	2, 250	24. 3	2, 500	26. 9	2, 960	3, 080
8	23. 0	2, 280	23. 5	2, 360	27. 4	3, 050	2, 950
9	23. 5	2, 360	24. 9	2, 600	28. 0	3, 160	3, 270
10	25. 6	2, 730	26. 0	2, 800	30. 6	3, 650	3, 900
11	26. 9	2, 960	31. 0	3, 730	34. 8	4, 560	3, 870
12	26. 9	2, 960	26. 5	2, 890	30. 2	3, 570	4, 180
13	27. 9	3, 140	30. 0	3, 530	32. 8	4, 120	3, 800
14	28. 0	3, 160	27. 0	2, 980	33. 8	4, 340	4, 790
15	28. 7	3, 290	29. 2	3, 380	33. 8	4, 340	4, 700
16	29. 4	3, 420	32. 2	3, 980	35. 8	4, 780	4, 200
Average	24. 1	2, 510	25. 3	2, 700	29. 6	3, 500	3, 540
Index	100	----------	105	----------	123	----------	----------

[1] Each rebound value on side is an average of 12 readings; values on top and bottom are averages of 5 readings.
[2] Estimated values taken from curve in figure 2.
[3] Results of strength tests on cylinders.

Table 2.—Rebound readings on concrete cylinders in dry condition and after 24 hours immersion in water [1]

Cylinder number	Average rebound reading, dry	Estimated compressive strength [2]	Average rebound reading, wet	Estimated compressive strength [2]
		P. s. i.		*P. s. i.*
1	26.8	2,940	25.0	2,620
2	27.7	3,100	27.3	3,040
3	27.8	3,120	26.9	2,960
4	28.3	3,220	26.0	2,800
5	28.7	3,280	26.9	2,960
6	29.4	3,420	26.3	2,860
7	34.7	4,540	30.5	3,630
8	35.1	4,620	32.8	4,120
9	35.8	4,780	32.5	4,050
10	37.6	5,170	34.4	4,470
Average	31.2	3,820	28.9	3,350

[1] Each value is an average of 12 readings. Approximate age of dry cylinders was 14 days.
[2] Estimated values taken from curve in figure 2.

Table 3.—Rebound readings on concrete cylinders tested at various ages

Age at test	Average rebound reading [1]	Estimated compressive strength [2]	Actual compressive strength [3]
Days		*P. s. i.*	*P. s. i.*
5	21.9	2,110	3,410
10	25.0	2,620	3,920
20	28.3	3,220	4,800

[1] Each rebound reading is an average of 12 readings on each of 4 cylinders.
[2] Estimated values taken from curve in figure 2.
[3] Results of strength tests on cylinders.

procedures or in materials used may account for this variation.

With few exceptions the individual rebound readings show very little variation from the average. A few readings were excluded because the hammer had probably been held against a piece of exposed aggregate or against a thin layer of mortar over a void.

The Swiss Hammer was also used on the piers of a bridge which were about 2½ years old. The average compressive strength of control test cylinders at the age of 28 days was reported as 4,500 p.s.i. The average estimated compressive strength of this concrete at 2½ years, as determined from the rebound readings given in table 6 and the curve in figure 2, was 5,660 p.s.i.

The reconstruction of a bridge which was about 38 years old offered an excellent opportunity to try the Swiss Hammer on concrete in place and on specimens obtained for tests in the laboratory. This concrete was in good condition. Swiss Hammer readings were taken at four locations on the vertical face of the hand rail of the bridge. The average rebound reading of 40.9, when checked against the curve in figure 2, indicates a compressive strength of 5,850 p.s.i.

Three prisms approximately 6 inches square and 9 inches high were sawed from the hand rail at about the same location where the rebound readings were taken. After these specimens had been prepared for tests of compressive strength, they were placed under a small load in the testing machine. Rebound readings of these specimens had an average value of 41.2, corresponding to an estimated compressive strength of 5,980 p.s.i. The actual compressive strength of the three prisms corrected for H/D (height over depth) averaged 5,470 p.s.i.

In two of the three trials of the Swiss Hammer on concrete structures, direct comparisons could be made between the actual compressive strength of test specimens representing the concrete and the compressive strength as determined from the rebound reading and use of the curve in figure 2. In both cases, the rebound reading indicated a strength 10 to 20 percent higher than was obtained by

Table 4.—Relation between rebound readings on concrete cylinders and beams prepared with two types of aggregate [1]

Gravel concrete				Stone concrete			
Rebound reading		Actual compressive strength of cylinders	Modulus of rupture of beams	Rebound reading		Actual compressive strength of cylinders	Modulus of rupture of beams
Side of cylinders [2]	End of beams [2]			Side of cylinders [2]	End of beams [2]		
		P. s. i.	*P. s. i.*			*P. s. i.*	*P. s. i.*
24.6	25.5	2,920	475	24.4	26.2	4,020	675
25.3	23.5	2,380	450	25.7	25.1	3,660	670
26.0	25.5	3,220	455	26.3	27.2	4,220	710
26.6	27.0	3,640	445	27.1	26.0	4,230	735
26.8	25.2	2,450	490	27.6	28.0	4,560	720
27.3	27.3	3,630	460	27.8	28.8	4,550	740
27.9	28.6	3,190	510	28.1	28.8	4,760	720
28.4	26.5	3,570	470	28.6	28.8	4,520	785
29.0	27.3	3,420	420	28.9	28.2	4,740	755
29.2	28.7	3,380	480	29.4	29.6	4,990	690
29.7	28.8	3,740	460	29.6	28.4	5,090	720
29.7	30.7	3,840	470	30.4	32.6	5,400	725
29.8	29.7	3,900	440	30.8	30.4	4,850	800
30.3	31.1	3,490	480	31.1	29.3	4,990	795
30.4	30.5	4,200	445	31.9	31.4	5,540	735
31.4	30.6	3,990	480	31.9	33.6	5,840	770
31.6	30.6	3,820	480	32.2	32.0	5,360	780
31.6	31.8	3,790	560	32.2	34.4	5,430	840
31.7	30.8	3,880	535	32.4	31.9	5,140	815
32.9	30.1	3,860	595	32.6	31.8	5,650	690
33.0	35.0	4,950	515	33.7	35.3	5,660	930
33.1	33.4	4,450	610	36.0	37.3	6,010	955
34.5	33.5	4,640	575	36.0	37.6	6,080	905
35.2	35.1	4,960	490	38.8	37.5	6,100	775
35.9	37.0	5,400	590	38.8	38.8	6,330	925
36.2	40.0	4,780	675	39.1	36.9	6,620	845
36.2	33.8	4,420	610	40.9	40.2	6,740	925
38.6	39.4	5,140	595	41.2	39.6	6,890	915
41.4	40.5	5,440	650	43.7	41.9	7,090	925
(31.2)	(30.9)	(3,950)	(515)	(32.3)	(32.3)	(5,350)	(790)

[1] Figures in parentheses are average values for all tests.
[2] Each value is an average of 10 or 12 readings.

Table 5.—Rebound readings obtained with Swiss Hammer in field tests of beams for a post-tensioned bridge

Beam No. 5, age 8 days	Beam No. 5, age 16 days	Beam No. 2, age 35 days	Beam No. 4, age 21 days	Beam No. 4,[1] age 29 days
REBOUND READINGS [2]				
35	44	31	38	41
36	[3] 47	39	38	42
39	44	40	38	40
37	43	41	36	39
37	38	38	37	40
[3] 44	37	37	35	38
34	44	39	40	40
41	36	38	35	38
36	37	40	40	38
34	36	37	36	41
33	35	[3] 48	38	39
34	38	41	[3] 28	40
35	41	36	40	41
33	37	36	38	37
39	42	40	38	42
33	33	36	38	39
35	36	36	43	37
36	34	36	39	38
31	39	36	34	43
[3] 51	37	35	40	43
37	42	[3] 45	38	47
30	34	34	[3] 47	37
30	38	34	38	37
38	41	34	38	44
33	34	37	[3] 48	45
35	34	38	35	38
37	37	37	42	39
31	38	-------	-------	-------
36	36	-------	-------	-------
40	39	-------	-------	-------
(35.2)	(38.1)	(37.0)	(38.0)	(40.1)
COMPRESSIVE STRENGTH [4] (p. s. i.)				
(4,640)	(5,280)	(5,040)	(5,260)	(5,720)
3,700	4,100	4,600	4,600	4,700
[5] 25	[5] 29	[5] 10	[5] 14	[5] 22

[1] Readings taken after beam had been stressed.
[2] Figures in parentheses are averages of all rebound readings.
[3] Readings not included in averages.
[4] Figures in parentheses represent estimated compressive strengths of concrete, whereas figures immediately below are the actual compressive strengths of the control cylinders.
[5] Percentage that estimated compressive strength exceeded actual compressive strength of control cylinders.

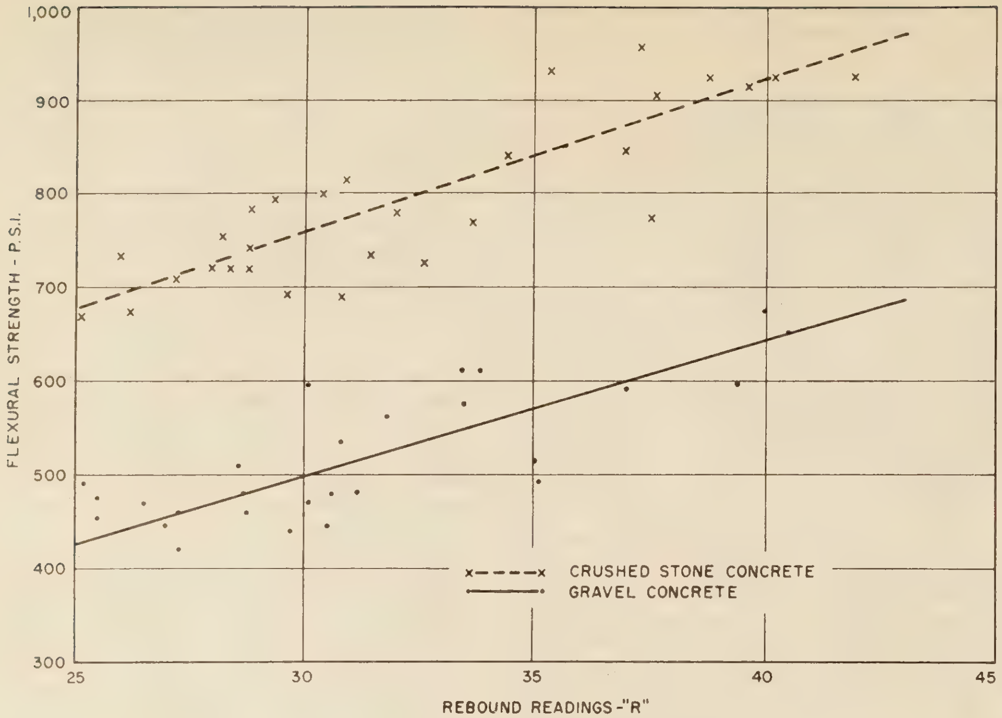

Figure 8.—Relation between flexural strengths and rebound readings on 6- by 6- by 21-inch concrete beams prepared with two types of aggregate.

Table 6.—Rebound readings obtained with a Swiss Hammer in field testing of bridge piers

Rebound readings [1] for—			
West pier	Center pier, east side	Center pier, west side	East pier
41	42	37	39
41	42	31	37
42	48	43	35
42	43	37	35
42	40	38	39
41	40	38	40
42	38	43	40
44	44	50	40
39	48	42	34
43	32	34	36
38	43	34	38
40	36	41	32
42	38	42	38
41	36	39	36
46	41	40	46
41	41	40	38

[1] The respective average rebound readings and estimated compressive strengths of concrete cylinders were as follows: west pier, 41.6 and 6,030 p. s. i.; center pier (east side), 40.8 and 5,870 p. s. i.; center pier (west side), 39.3 and 5,550 p. s. i.; and east pier, 37.7 and 5,200 p. s. i.

testing the specimens. It is apparent that the curve given in figure 2 should be used with reservations. For the best determinations, a curve should be prepared showing the relation between strength and rebound reading for concrete of the same type and composition as that to be inspected.

Considerable wear was found on the face of the striking rod of the Swiss Hammer after making the tests described in this article. Check tests made in the laboratory showed little effect on the indicated reading from this wear. However, for extended use it would be desirable to have a harder wearing surface on the face of the hammer.

Factors Affecting Results of Tests Using Swiss Hammer

In using the Swiss Hammer, there are a number of factors which affect the readings. The following conditions should be considered in interpreting the results:

Condition of the surface of the concrete.—Readings taken on a polished surface are high, whereas readings taken on a rough surface (such as a broomed surface) are low.

Moisture condition on the concrete.—Concrete in a moist condition gives a lower reading than concrete in a dry condition.

Type of coarse aggregate.—The type of coarse aggregate used and possibly the composition of the concrete affect the amount of rebound.

Value of the Swiss Hammer

The Swiss Hammer provides a quick and inexpensive method for checking the uniformity and estimating the strength of hardened concrete. It is not intended as a substitute for control test cylinders, nor is it intended to give an accurate measure of the compressive strength of the concrete. It is valuable for use in the field for "trouble shooting" to determine whether test cores are needed and where they should be drilled. It may be used to determine the rate of increase in strength of concrete and to determine when forms can be removed or loads applied. It may also be used to estimate the extent of damage to structures caused by freezing or fire, and to judge the quality of the concrete in old structures.

Figure 9.—Swiss Hammer being used in field testing.

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Wind Tunnel for Aerodynamic Testing of Section Models of Suspension Bridges

3Y THE DIVISION OF PHYSICAL RESEARCH
3UREAU OF PUBLIC ROADS

A WIND TUNNEL for aerodynamic testing of section models of suspension ridges has been constructed at the Bureau of 'ublic Roads Research Station near Langley, 'a. It was designed by personnel of the 3ureau of Public Roads after consulting with fficials of the Aeronautics Section of the Vational Bureau of Standards. Much of the perating equipment and instrumentation was lso built in the Bureau's shops. A few ıstruments are yet to be provided.

The failure of the Tacoma Narrows Bridge n November 7, 1940, in a wind of moderate elocity made it evident to engineers conerned with the design of suspension bridges hat it would be necessary to make a thorough ıvestigation of the performance of bridges of his type when subjected to wind action, if ıture bridges were to be designed with ssurance that they would be safe from atastrophic motion. Upon the request of a epresentative group of these engineers, the 'ommissioner of Public Roads organized the .dvisory Board on the Investigation of Susension Bridges for the purpose of initiating nd correlating such investigations. Followıg the recommendation of the Advisory 3oard, the Bureau engaged in several investiations in cooperation with the Washington 'oll Bridge Authority, the University of Vashington, the Golden Gate Bridge and Iighway District, the American Institute of teel Construction, and the Oregon State Iighway Department.

Previous Studies

The studies that were made in cooperation 'ith the Washington Toll Bridge Authority nd the University of Washington accomlished the objectives of determining quite atisfactorily the nature of the wind action 'hich caused the failure of the original girdertiffened Tacoma Narrows Bridge and indiating the design provisions needed to insure he stability of the new truss-stiffened strucure which has replaced it. These studies also howed how a scaled section model, representıg a limited length of a suspension bridge and ıounted in the wind tunnel on properly esigned springs, may be used to show the erodynamic characteristics of the bridge 'ithout the necessity of testing a complete cale model.

The studies were extended to cover a numer of other girder- and truss-stiffened bridges ıcluding the Golden Gate Bridge as originally uilt. The cooperative observations on the Golden Gate Bridge provided the opportunity to correlate the behavior of a bridge in the field and its model in the wind tunnel. Analytical studies by the late Dr. Friedrich Bleich,[1] made possible through the cooperation with the American Institute of Steel Construction, provided rational correlation with both the laboratory tests and the field observations.

Major design features which provide aerodynamic stability in the new Tacoma Narrows Bridge are (1) the bottom lateral system which materially increases the torsional stiffness of the structure and raises the frequency of any torsional oscillation which might occur, and (2) the grated slots in the roadway which break up the action of wind forces tending to excite "flutter," a form of coupled oscillation of bending and torsional motions. The studies have shown these design features to be effective in stabilizing other truss-stiffened suspension bridge sections but the nature and degree of the benefit varies. For this reason, tests and studies on a specific design are considered advisable in order to establish the effectiveness and optimum proportions of the stabilizing features. Brief studies on means for stabilizing girder-stiffened sections have not as yet yielded full solutions, and additional data on this factor are needed.

The wind tunnel at the University of Washington, which was specially designed for testing suspension bridge models, was of temporary construction and will not be available for continued studies. The Advisory Board on the Investigation of Suspension Bridges recommended that the Bureau of Public Roads provide facilities to continue the testing of section models. A nationwide survey indicated that it would cost less to provide and operate a special wind tunnel in the laboratory of the Bureau than to make the test-section adaptations and other modifications required to equip an ordinary wind tunnel for this special type of testing. The use of an ordinary wind tunnel would result in an intermittent test program because of the necessity of fitting the program into an already crowded working schedule.

Mr. George S. Vincent, who for 11 years represented the Bureau of Public Roads at the University of Washington while the cooperative wind-tunnel studies were being made there, will be in charge of the Bureau's research program on bridge section models. Mr. Vincent is a coauthor of the report *Aerodynamic Stability of Suspension Bridges with Special Reference to the Tacoma Narrows Bridge*, which was published as a result of the studies made at the University of Washington.

Description of Wind Tunnel

The type of testing required for bridge section models utilizes only relatively low wind velocities but demands fine control of the wind and more than ordinary precision in the measurement of wind velocity. Provision for varying the direction of the wind in the vertical plane is also required. The test section of the new tunnel is open and has a nozzle 6 feet square. Control of the vertical angle of the wind is accomplished by rotating the nozzle about the cylindrical pressure chamber. The model, mounted on springs designed to reproduce the proper scaled frequency of oscillation, can be moved vertically and longitudinally on its supporting mechanism in order to place it in the desired position in the windstream.

Wind velocities up to about 50 feet per second are provided by a 60-inch double inlet fan driven by a 50-horsepower direct current motor which can be controlled continuously from creep to a maximum speed of about 500 r. p. m. The airstream from the fan passes through a series of 40-mesh, stainless steel diffusing screens in a duct which diverges vertically and laterally and leads to a 13- by 13-foot pressure chamber, the size being restricted by the headroom available. This produces a very uniform flow of air at the nozzle. A double screen, pivoted at the center of the pressure chamber, can be rotated as may be required to correct the flow when the vertical angle of the windstream is varied. Finely controlled relief or bypass openings just downstream from the fan afford additional control of the wind velocity.

The features described are shown in figure 1. Figure 2 is a photograph showing a section model of the new Tacoma Narrows Bridge mounted on springs in front of the nozzle of the wind tunnel. The carriage for the model, rolling on the tracks which support its ends, can be moved longitudinally with respect to the axis of the windstream, and the entire assembly can be fixed at any desired elevation on the four supporting tubular columns. At the far left, on a concrete pedestal, can be seen one of the axle bearings on which the nozzle may be rotated vertically by raising or lowering its outlet end.

[1] *The mathematical theory of vibration in suspension bridges*, by Bleich, McCullough, Rosecrans, and Vincent. Chapters 3-8. Bureau of Public Roads, 1950.

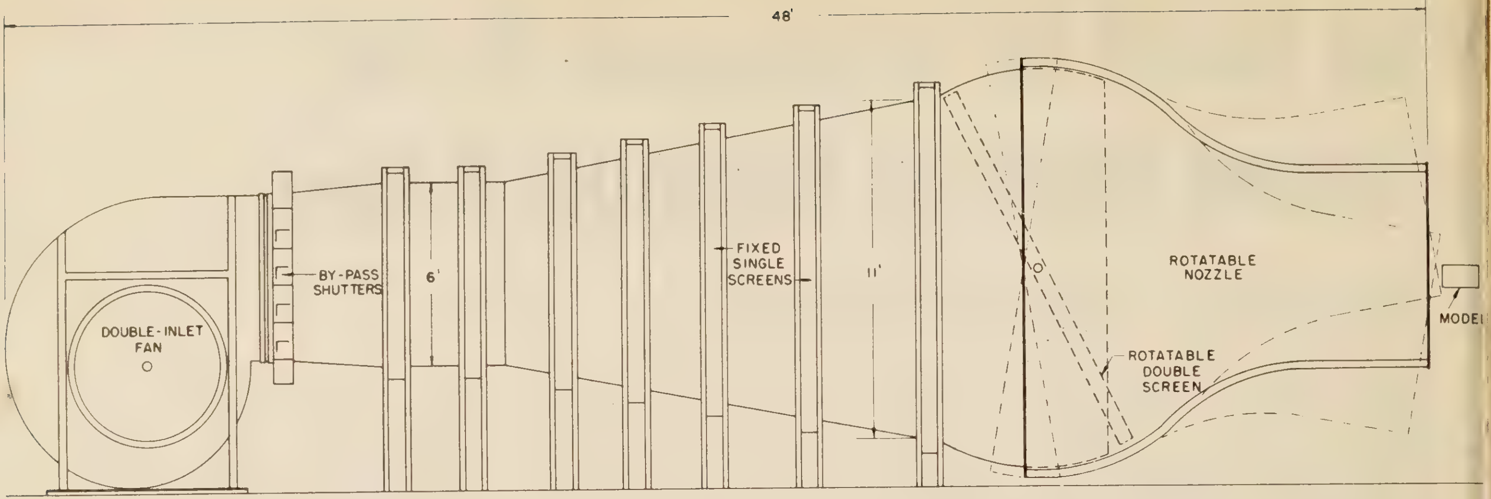

Figure 1.—Side elevation giving shape and principal dimensions of wind tunnel.

Figure 2 also shows two pitot-static tubes mounted at the sides of the nozzle. These can be moved to cover any part of the airstream at the nozzle and for some distance downstream. They are connected to the manometer at the left which is read by a micrometer-mounted telescope. The console in the foreground carries the controls for the fan motor and for the bypass shutter mechanism as well as meters indicating the rate of rotation of the fan, the amperage of the drive motor, and the amperage of the motor-generator converter. Space is also provided for other meters and instruments to be used in the tests.

Following tests of the characteristics and control of the windstream itself, the tunnel will be used to repeat tests on section models previously tested in the wind tunnel at the University of Washington in order to determine whether they are influenced by individual "tunnel effects" such as are commonly observed in aeronautical research. High on the list of subsequent testing will be studies to discover means for stabilizing girder-stiffened suspension bridges.

The wind tunnel can be used to study a variety of problems involving wind-excited vibration of structures such as overhead signs on expressways, for example. For tests requiring wind velocities exceeding 50 feet per second, the nozzle can be choked with false walls, provided the dimensions of the model permit reducing the height or width of the test section.

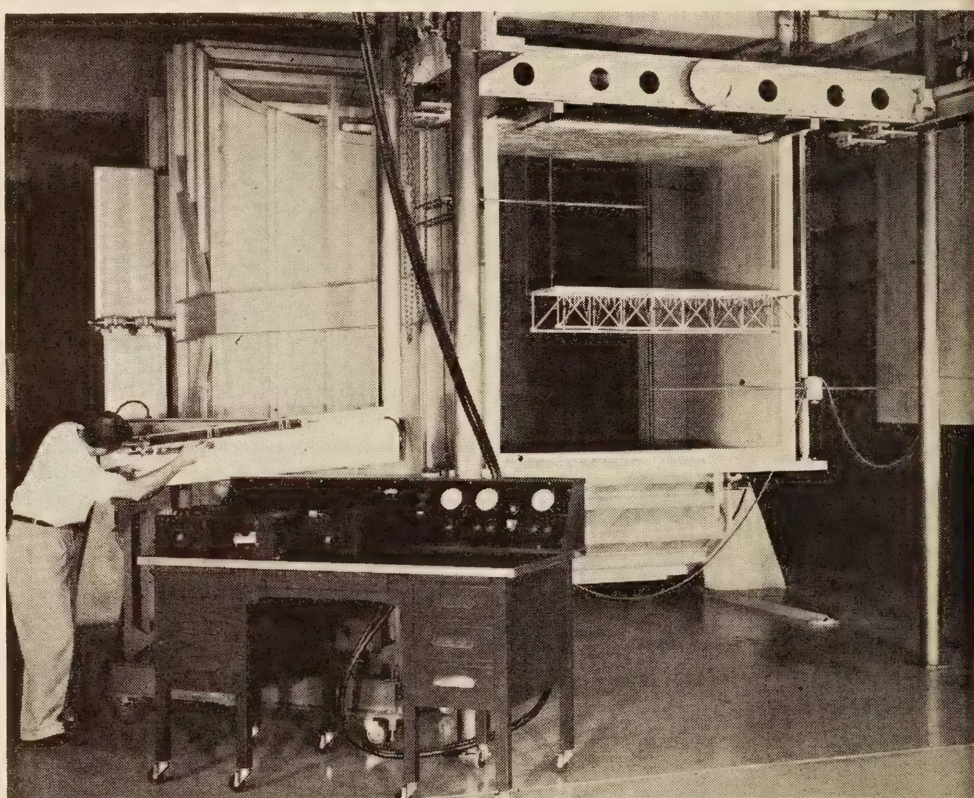

Figure 2.—Nozzle of wind tunnel with a bridge section mounted in front. The console i the foreground provides controls for the operation of the wind tunnel.

Power Shovel Productivity: A Motion Picture

The Bureau of Public Roads, U. S. Department of Commerce, recently announced the release of a new motion picture, *Power Shovel Productivity*. The film, based on extensive studies conducted by Public Roads, highlights the job conditions that determine the yardage output of power shovels on highway grading work, and demonstrates how production is affected by the speed of dipper cycle, size of dipper load, and frequency and duration of minor delays.

The motion picture is a 16-mm. sound and color film with a running time of 30 minutes. Prints may be borrowed for showings by any responsible organization by request addressed to Visual Education, Bureau of Public Roads, Washington 25, D. C. There is no charge except for the express or postage fees. At the present time, bookings are solid through the summer months. Requests for the film should be sent well in advance of the desired showing and alternate dates for showing should be given if possible. Immediate retu after each showing is necessary, so that requested bookings may be fulfilled.

Prints of the film may be purchased $111.88 per copy, the price including film, re can, and shipping container, and posta within the United States. Inquiries shou be addressed to Visual Education, Bureau Public Roads, Washington 25, D. C. Pa ment should *not* be sent with the inquiry.

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U.S. DEPARTMENT OF COMMERCE - BUREAU OF PUBLIC ROADS

STATUS OF FEDERAL-AID HIGHWAY PROGRAM

AS OF APRIL 30, 1958

(Thousand Dollars)

STATE	UNPROGRAMMED BALANCES 1/	ACTIVE PROGRAM: PROGRAMMED ONLY			CONTRACTS ADVERTISED, CONSTRUCTION NOT STARTED			PROJECTS UNDER WAY			TOTAL		
		Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles	Total Cost	Federal Funds	Miles
Alabama	$33,229	$58,583	$47,592	345.0	$20,920	$13,543	150.9	$87,658	$57,669	929.8	$167,161	$118,804	1,425.7
Arizona	14,918	26,367	23,386	165.4	7,515	6,929	48.2	39,095	33,928	203.1	72,977	64,243	416.7
Arkansas	27,294	44,221	35,452	522.6	20,401	15,235	133.5	49,691	35,707	414.3	114,313	86,394	1,070.4
California	66,483	52,836	30,964	218.3	29,514	24,178	32.7	589,436	273,631	424.9	671,786	328,773	675.9
Colorado	44,557	12,035	9,048	92.5	2,568	2,345	4.2	62,150	44,131	347.3	76,753	55,524	444.0
Connecticut	43,642	8,719	4,990	10.2	13,446	8,625	4.8	46,306	32,796	36.0	68,471	46,411	51.0
Delaware	24,456	11,309	8,607	23.8	3,624	2,315	19.2	13,997	7,953	64.7	28,930	18,875	107.7
Florida	20,116	57,509	44,321	268.4	10,776	6,827	54.1	75,028	54,367	333.9	143,313	105,515	656.4
Georgia	20,323	110,541	81,639	622.3	11,956	7,294	69.5	138,499	87,447	1,050.1	260,996	176,380	1,741.9
Idaho	38,174	16,873	14,159	97.1	2,874	2,436	33.6	28,267	20,585	222.9	48,014	37,180	353.6
Illinois	54,172	101,204	68,723	731.7	86,053	61,446	123.2	224,962	166,957	724.5	412,219	297,126	1,579.4
Indiana	78,480	71,060	53,191	243.8	56,045	33,868	359.8	61,605	40,203	249.1	188,710	127,262	852.7
Iowa	41,321	45,437	33,765	598.6	29,431	23,392	184.0	65,756	44,783	1,201.7	140,624	101,940	1,984.3
Kansas	39,363	26,804	20,691	814.7	24,240	16,687	303.3	65,478	45,363	1,326.6	116,522	82,741	2,444.6
Kentucky	51,582	48,830	35,057	127.6	2,717	1,861	11.6	76,183	52,935	286.1	127,730	89,853	425.3
Louisiana	26,862	53,861	35,769	336.0	51,577	37,250	207.3	68,028	41,475	360.3	173,466	114,494	903.6
Maine	26,775	11,034	8,649	12.8	4,280	3,847	2.6	26,485	17,892	103.3	41,799	30,388	118.7
Maryland	11,231	29,548	17,734	153.5	23,891	15,166	35.9	79,398	57,895	199.5	132,837	90,795	388.9
Massachusetts	32,610	46,479	35,991	21.2	44,136	24,215	27.7	106,280	74,489	62.7	196,895	134,695	111.6
Michigan	52,302	96,798	72,687	660.2	39,119	26,125	226.7	139,986	101,652	410.1	275,903	200,464	1,297.0
Minnesota	67,659	13,538	10,774	53.5	12,575	7,366	135.5	101,679	76,467	953.2	127,792	94,607	1,142.2
Mississippi	18,540	50,293	36,584	699.7	21,463	15,468	167.5	75,455	52,932	876.4	147,211	104,984	1,743.6
Missouri	41,708	42,005	27,215	1,339.1	21,242	16,266	46.7	122,613	83,090	1,075.0	185,860	126,571	2,460.8
Montana	68,913	12,166	8,314	171.2	4,231	3,565	24.3	49,531	36,721	439.3	65,928	48,600	634.8
Nebraska	63,825	10,511	6,684	229.0	11,304	5,967	154.7	64,512	42,337	1,152.6	86,327	54,988	1,536.3
Nevada	36,173	8,091	7,506	71.1	1,163	1,014	18.6	30,461	27,919	194.9	39,715	36,439	284.6
New Hampshire	21,370	4,612	2,529	16.8	1,918	1,405	3.4	25,979	17,832	71.0	32,509	21,766	91.2
New Jersey	88,281	18,620	10,152	64.1	19,977	15,162	10.6	83,060	58,711	48.0	121,657	84,025	122.7
New Mexico	13,309	10,264	9,115	37.0	6,389	5,576	65.7	49,860	40,298	339.8	66,513	54,989	442.5
New York	146,649	54,314	37,420	115.4	61,156	45,503	56.1	534,203	341,546	409.1	649,673	424,469	580.6
North Carolina	67,463	51,669	41,957	225.7	10,850	7,487	134.0	96,370	63,749	857.7	158,889	113,193	1,217.4
North Dakota	23,056	26,819	19,484	1,254.5	19,245	14,901	315.9	28,887	19,656	1,148.4	74,951	54,041	2,718.8
Ohio	39,434	48,674	26,514	118.4	41,561	31,755	73.1	278,927	200,621	355.1	369,162	258,890	546.6
Oklahoma	17,030	73,096	52,317	780.3	19,306	15,719	93.7	66,573	45,212	620.2	158,975	113,248	1,494.2
Oregon	34,360	7,904	5,422	76.3	12,145	10,093	67.9	45,039	35,766	248.5	65,088	51,281	392.7
Pennsylvania	105,908	109,426	75,110	216.2	77,406	52,039	95.0	263,564	177,142	369.7	450,396	304,291	680.9
Rhode Island	12,065	8,030	4,129	8.2	9,309	7,981	10.7	22,630	16,675	16.2	39,969	28,785	35.1
South Carolina	24,304	52,282	39,126	646.2	14,588	12,057	90.3	51,673	32,249	779.2	118,543	83,432	1,515.7
South Dakota	11,163	61,736	49,289	687.2	1,197	668	166.4	30,284	22,619	575.5	93,217	72,576	1,429.1
Tennessee	36,033	68,130	52,992	480.0	13,893	9,510	63.1	120,565	77,318	741.0	202,588	139,820	1,284.1
Texas	86,634	113,180	94,772	702.0	52,091	40,005	272.5	240,730	172,380	2,071.3	406,001	307,157	3,045.8
Utah	12,502	29,886	27,114	194.9	4,023	3,448	18.5	29,023	25,110	130.3	62,932	55,672	343.7
Vermont	7,820	20,702	18,171	27.5	401	200	.9	22,880	16,412	61.9	43,983	34,783	90.3
Virginia	21,469	61,722	48,678	195.0	2,224	1,115	11.0	105,182	77,165	324.8	169,128	126,958	530.8
Washington	33,194	26,420	18,761	218.9	13,553	9,343	44.2	72,985	55,544	272.2	112,958	83,648	535.3
West Virginia	40,738	50,091	36,764	89.4	5,340	2,745	15.0	40,876	23,130	74.8	96,307	62,639	179.2
Wisconsin	83,004	31,144	20,382	368.2	16,196	11,464	154.3	61,366	40,623	430.8	108,706	72,469	953.3
Wyoming	12,251	19,604	16,281	84.2	1,196	1,126	2.1	48,838	40,786	336.0	69,638	58,193	422.3
Hawaii	4,472	9,250	4,625	16.7	1,410	650	1.8	2,689	1,341	4.1	13,349	6,616	22.6
District of Columbia	17,665	17,760	14,138	9.2	2,380	1,247	2.1	28,783	21,809	5.0	48,923	37,194	16.3
Puerto Rico	10,308	16,087	7,785	25.2	2,006	986	3.8	19,310	8,914	58.1	37,403	17,685	87.1
Alaska	7,756	15,658	14,313	254.9	787	787	20.5	11,838	10,331	314.8	28,283	25,431	590.2
TOTAL	2,022,946	2,073,732	1,526,832	15,541.7	967,608	686,202	4,372.7	4,870,653	3,254,263	24,305.8	7,911,993	5,467,297	44,220.2

1/ Includes additional funds authorized by Federal-Aid Highway Act of 1958, apportioned April 16, 1958.