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Mass Transit in the Utilization of City Streets

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● MASS TRANSPORTATION is recognized as a highly desirable component of the over-all transportation scheme in urban centers. In recent years mass transit systems have experienced a decline in patronage while the number of persons using privately owned automobiles has represented an increasingly larger percentage of the people transported over city streets. The overcrowding of streets is often attributed to this decline in mass transit patronage, and fears have been expressed that cities, and more particularly their transit companies, are facing economic ruin unless the trend is arrested. Subsidization has been proposed, and in several cases resorted to as a means of maintaining mass transit service.

The purpose of the street system is to serve the public, and decision as to how this service can be most satisfactorily and economically provided rests with the highway user, taken collectively. The increased use of private automobiles at the expense of transit patronage is merely a registration of highway user attitudes regarding this decision. However, the expansion in automobile usage has thus far taken place on streets that have undergone only minor change or improvement through the years, and the outlay for capital improvements for street systems has not approached the rate of growth of private automobile usage.

Heavy expenditure for street improvement is a prerequisite to the unabated growth of vehicular traffic. Therefore, the full impact of road-user costs has not been felt by the private automobile user, and the preference which he has thus far exhibited might be altered if he could be apprised of his share of the burden of street improvement costs.

A thorough investigation of the economics of mass transit versus private automobile usage of city streets would be very far reaching and would cover all of the various aspects of user costs, convenience and comfort factors, time savings or losses, and other operational economies. As one phase of an economic investigation, this study compares the relative efficiencies of mass transit vehicles and private automobiles in their manner of utilizing street space and moving people.

A STANDARD FOR MEASURING EFFICIENCY

Any measure of efficiency must consider the space occupied per person in the moving traffic stream, and the length of time that the space is occupied in traveling a given distance. Space per person may be expressed as the space occupied in the traffic stream by a vehicle of a particular type, divided by the number of persons carried by the vehicle. The length of time that the space is occupied may be determined by dividing the distance traveled by the over-all speed of the vehicle. Thus, for comparing efficiencies of different modes of transportation, there are three elements to be considered for each type of vehicle if conclusive results are to be obtained. These are the speed of the vehicle, the space occupied by the vehicle in the traffic stream, and the carried load. These three variables were investigated for automobiles, buses and streetcars in the Washington, D.C. metropolitan area, and for automobiles and trolley coaches in Atlanta, Georgia.

BUS OPERATION ON SURFACE STREETS

The investigation of bus operation included studies of travel speeds and of passengers carried by buses in normal operation during various periods of the day, and of the effect that buses have on street capacity. The speed and loading studies covered a number of bus lines in the Washington metropolitan area. Observers posted on buses compiled complete records showing the number of persons on board between stops, the number of persons boarding and alighting at each stop, the travel time between stops,

TABLE 1

AVERAGE NUMBER OF PASSENGERS CARRIED BY BUSES

Time	Average load (persons per bus)		
	Downtown	Intermediate	Outlying
Morning peak	30.1	47.1	25.6
Afternoon peak	48.5	53.4	36.0
Peak hour avg.	39.3	50.2	30.8

TABLE 2

AVERAGE TRAVEL TIME FOR BUSES

Time	Travel time (min per mi)		
	Downtown	Intermediate	Outlying
Morning peak	10.06	6.17	5.15
Afternoon peak	9.18	6.52	5.42
Peak hour avg.	9.62	6.35	5.28

from traffic signals, and they represent the travel time for buses that were inbound during the morning peak and outbound during the afternoon peak.

Space Occupied by a Bus

In the study of the effect that buses have on street capacity, the maximum rates of traffic flow (possible capacities) for selected intersections in the Washington metropolitan area were determined by counting the numbers of vehicles entering each intersection from one of its approaches when the traffic demand was equal to or greater than the capacity of that approach. The traffic count for each signal cycle was classified as to the extent of interference by buses; that is, no interference, interference by one bus, by two buses, etc. The capacity of the intersection approach without bus interference was compared to its capacity with bus interference, and a determination was made of the number of automobiles displaced by one bus in the traffic stream. Satisfactory data were obtained for 16 intersection approaches, 11 on arterial streets and 5 on secondary or feeder streets. This study was conducted in June, 1953.

The amount of space in a traffic stream

the length of time spent in loading and unloading operations, and the cause and duration of delays.

The study extended over a period of five days, Monday through Friday, during the spring of 1950. Information was obtained on 283 bus trips (one way) divided about equally between the morning rush period, the afternoon rush period, and the off-peak or base period. Thirteen operating routes were studied.

Average Carried Load

The average loads carried by the buses during peak periods are summarized in Table 1. These buses were inbound during the morning peak and outbound during the afternoon peak. The number of persons shown is the average per bus, weighted according to passenger-miles traveled.

Operating Speed

The travel time for buses is shown in Table 2. The figures in Table 2 include minor traffic delays and delays resulting

TABLE 3

AVERAGE NUMBER OF AUTOMOBILES DISPLACED BY ONE BUS IN A STREAM OF TRAFFIC

Time	Number of Automobiles Displaced	
	Arterial Street	Secondary Street
Morning peak	3.9	1.6
Afternoon peak	2.7	1.7
Peak hour avg.	3.3	1.6

TABLE 4

AVERAGE TRAVEL TIME FOR AUTOMOBILES ON STREETS USED BY BUSES

Time	Travel time (min per mi)		
	Downtown	Intermediate	Outlying
Morning peak	5.79	3.94	2.84
Afternoon peak	6.17	3.86	3.11
Peak hour avg.	5.98	3.90	2.96

TABLE 5

RELATIVE EFFICIENCIES OF AUTOMOBILES AND BUSES IN THE UTILIZATION OF STREET SPACE AND MOVEMENT OF PEOPLE DURING PEAK HOURS OF TRAFFIC MOVEMENT ON SURFACE STREETS

Area	Type of Vehicle	Travel Time (min per mi)	Carried Load (persons per veh)	Space per Vehicle (auto. units)	Relative Efficiency
Downtown	Automobiles	6.0	2.0	1.0	1.0
	Buses	9.7	39.3	3.3	3.7
Inter-mediate	Automobiles	3.9	2.0	1.0	1.0
	Buses	6.4	50.2	3.3	4.6
Outlying	Automobiles	3.0	2.0	1.0	1.0
	Buses	5.3	30.8	3.3	2.6

that a bus occupies varies with the street width, the number of passengers loading and unloading, the gradient of the street, parking conditions, and a number of less important factors. The 11 arterial-type streets on which studies were conducted were from 40 to 60 ft wide. All but one of these intersection approaches had near-side bus stops and the exception had a far-side stop. None of the study sites was in the central business district although some were on the fringe of the downtown area. Most of the locations were in intermediate-type areas with a few being in the outlying suburban districts. Parking was not permitted on any of the arterials during the course of the studies and all bus traffic was straight through. The interchange of passengers at the bus stops was only moderately heavy. Studies were conducted during morning and afternoon hours of peak traffic movement. With such a notable variation in the conditions at the various study sites, a corresponding variation in results is to be expected. On the average arterial, one bus was found to have a displacement equivalent to 3.3 automobiles (Table 3). The range in the passenger-car equivalent of one bus was from 2.1 to 6.0. For the five secondary sites the average bus was found to be equivalent to 1.6 automobiles with a range of from 1.0 to 1.9. An appreciable difference was observed in the bus equivalent between morning and afternoon periods on the arterial streets, the former being about one and one-half times the latter, on an average.

The difference between the morning and afternoon values is presumed to be a result of differences in the conditions at the study sites and not a result of any difference in the characteristics of bus operations for the two time periods. The average of the values for morning and afternoon on arterial streets will be used in subsequent comparisons of vehicle efficiencies.

Automobile Operation on Routes Used by Buses

For automobiles the space per vehicle in the traffic stream has been taken as unity for purposes of this analysis. The two other variables, namely, travel time and the average load for automobiles, were determined by field investigation. The average number of persons in automobiles was 1.97 persons per vehicle.

Travel time for automobiles was determined by driving a test car over the routes that were covered in the study of buses. On each of these routes, 9 trips were made with a test car driven at a speed which the driver thought was representative of the average speed of traffic. Three trips were made during the morning rush hour (not necessarily on the same day), 3 during the afternoon rush period, and 3 during the off-peak period. Results of these tests are shown in Table 4. The travel time shown for the morning peak is for inbound trips only, while outbound trips only are represented by the figures for the afternoon peak.

Relative Efficiencies of Automobiles and Buses on Surface Streets

Table 5 compares automobiles with buses in their travel time, carried load, the space per vehicle in the traffic stream, and relative efficiencies. Travel time, carried load, and space per vehicle are average values for the morning peak hour in the inbound direction and the afternoon peak hour in the outbound direction. The relative efficiencies for the two types of transportation, automobiles and buses, are shown in the right-hand column, using the automobile as unity as a basis for comparison. In the downtown area, 39.3 passengers in one bus occupy as much space in the traffic stream as 6.6 passengers in 3.3 automobiles, a ratio of 5.95 to 1 in favor of the bus. These bus passengers occupy their space 1.62 times as long as the automobile occupants in traveling any given distance, so the efficiency of the bus is thereby reduced by the speed differential in the ratio of 1 to 1.62, or to 62 percent. The resultant efficiency of the

TABLE 6

RELATIVE EFFICIENCIES OF AUTOMOBILES AND BUSES IN THE UTILIZATION OF STREET SPACE AND MOVEMENT OF PEOPLE DURING PEAK HOURS OF TRAFFIC MOVEMENT ON AN URBAN FREEWAY

Type of Vehicle	Travel Time (min per mi)	Carried Load (persons per veh)	Space per Vehicle (auto. units)	Relative Efficiency
Automobiles	1.8	1.7	1.0	1.0
Buses	2.4	27.9	1.7	7.2

bus in the downtown area during peak hours is 3.7 times that of the average automobile. In the intermediate area the bus is 4.6 times as efficient as the automobile, and in the outlying area the bus is 2.6 times as efficient as the automobile.

BUS OPERATION ON FREEWAYS

The average load and rate of travel for diesel-powered buses while operating in express service on a freeway were determined as one part of a study of mass transit operation in Atlanta in 1955. The bus route traverses the northeast leg of the Atlanta expressway system, which is a freeway with full control of access. There are no intermediate bus stops between the point where the route enters the freeway in the residential district and the point of exit in the downtown area.

During the morning peak period, the average bus on the expressway carried 31.3 passengers and traveled at the rate of one mile in 2.5 minutes. During the afternoon peak period the average load was 24.5 passengers, and the rate of travel was one mile in 2.4 minutes. For the morning and afternoon combined the average express bus carried 27.9 passengers and the average rate of travel was one mile in 2.4 minutes.

Automobiles using this portion of the Atlanta expressway carried an average of 1.7 persons during the morning and afternoon peak periods and traveled at the rate of one mile in 1.8 minutes.

The number of buses in service on this route was too small to permit a reliable determination of the space occupied by a bus in a stream of expressway traffic. The Shirley Memorial Highway in Arlington, Virginia, a freeway in the Washington metropolitan area, afforded a more satisfactory location for comparing the space occupied by an automobile with that occupied by a bus.

An extensive study by the Bureau of Public Roads revealed that, for any given volume of traffic, the average bus on the Shirley Highway occupies a time-gap in the traffic stream which is 1.7 times that for the average automobile. The study covered the operation of 658 buses and more than 75,000 automobiles.

Relative Efficiencies of Automobiles and Buses on a Freeway

By using the travel time and carried load for automobiles and buses as found on the Atlanta expressway, and the space occupied by a bus as found on the Shirley Highway, buses on freeways are found to be 7.2 times as efficient as automobiles in the utilization of street space and moving people (Table 6).

TABLE 7

AVERAGE NUMBER OF PASSENGERS CARRIED BY TROLLEY COACHES

Time	Average Load (persons per trolley coach)		
	Downtown	Intermediate	Outlying
Morning peak	50.4	51.7	38.9
Afternoon peak	47.0	52.5	40.1
Peak hour avg.	48.7	52.1	39.5

TABLE 8

AVERAGE TRAVEL TIME FOR TROLLEY COACHES

Time	Travel time (min per mi)		
	Downtown	Intermediate	Outlying
Morning peak	10.66	5.55	4.22
Afternoon peak	12.19	6.38	4.31
Peak hour avg.	11.42	5.96	4.26

TROLLEY COACH OPERATION

The procedures used in making studies of bus operation on surface streets in Washington were employed in a study of trolley coaches in Atlanta. The study was made in February and March 1955, and the results are summarized in Tables 7, 8, and 9. Table 7 shows the average number of passengers carried by trolley coaches, and Table 8 shows the average rate of travel by these vehicles. Table 9 shows the average number of automobiles displaced by one trolley coach on different types of streets during the morning peak and during the afternoon peak, from studies at 11 intersections. The similarity between the displacement of trolley coaches on arterial streets in Atlanta and of gaso-

TABLE 9

AVERAGE NUMBER OF AUTOMOBILES DISPLACED BY ONE TROLLEY COACH IN A STREAM OF TRAFFIC

Time	Number of Automobiles Displaced	
	Arterial Street	Secondary Street
Morning peak	4.0	3.8
Afternoon peak	2.6	3.8
Peak hour avg.	3.3	3.8

TABLE 10

AVERAGE TRAVEL TIME FOR AUTOMOBILES ON STREETS USED BY TROLLEY COACHES

Time	Travel time (min per mi)		
	Downtown	Intermediate	Outlying
Morning peak	8.19	5.29	3.69
Afternoon peak	8.62	5.90	4.16
Peak hour avg.	8.36	5.60	3.92

TABLE 11

RELATIVE EFFICIENCIES OF AUTOMOBILES AND TROLLEY COACHES IN THE UTILIZATION OF STREET SPACE AND MOVEMENT OF PEOPLE DURING PEAK HOURS OF TRAFFIC MOVEMENT ON SURFACE STREETS

Area	Type of Vehicle	Travel Time (min per mi)	Carried Load (persons per veh)	Space per Vehicle (auto. units)	Relative Efficiency
Downtown	Automobiles	8.4	1.7	1.0	1.0
	Trolley coaches	11.4	48.7	3.3	6.3
Intermediate	Automobiles	5.6	1.7	1.0	1.0
	Trolley coaches	6.0	52.1	3.3	8.7
Outlying	Automobiles	3.9	1.7	1.0	1.0
	Trolley coaches	4.3	39.5	3.3	6.3

line-powered buses on arterial streets in Washington (Table 3) is very striking. Trolley coaches as well as buses seem to reduce the capacity of arterial streets to a considerably greater extent during the morning period of peak traffic movement than during the afternoon peak. The reason for this is not clear. As in the case for buses, the average of the morning and afternoon values for the space occupied by trolley coaches on arterial streets is used in comparing vehicle efficiencies.

Automobile Operation on Routes Used by Trolley Coaches

At the time that trolley coaches were carrying the number of passengers shown in Table 6, automobiles using the same streets in Atlanta were carrying an average of 1.7 people. The average travel times for automobiles on these same streets are shown in Table 10.

Efficiencies of Automobiles and Trolley Coaches Compared

The three variables used in this analysis to compare efficiencies of the various types of vehicles are shown in Table 11 for automobiles and trolley coaches. The figures are average values for morning and afternoon peak periods. Relative efficiencies are shown in the right-hand column of this table. In the downtown area, for example, trolley coaches are shown as being 6.3 times as efficient as automobiles. Their speed is 0.7 that of automobiles; their load is 28.7 times that of automobiles; and their space in the traffic stream is 3.3 times that of an automobile. In the intermediate area the trolley coach is 8.7 times as efficient as the automobile, and in the outlying area the relative efficiency of the trolley coach is 6.3, the same as in the downtown area.

STREETCAR OPERATION

The procedure for studying streetcar operation differed in several respects from that employed for buses and trolley coaches. The major difference, however, was that vehicles and passengers were counted as they passed fixed points along the routes rather than by having observers on the vehicles. The majority of the study sites chosen were in the downtown business district, with the remainder being in the area immediately adjacent thereto. Study sites were selected at loading platforms where automobiles and streetcars were each allotted a specific portion of the street separate and distinct from that available to the other. Attempt was made to choose locations where conditions were most favorable for mass transit operations; that is, along those routes with the heaviest traffic and highly patronized transit service.

A total of 10 sites were studied and, with one exception, these sites happened to be on important automobile routes. However, transit traffic was the primary consideration governing their selection.

Studies were conducted during morning and afternoon hours of peak traffic on four weekdays in April 1952. Information collected consisted of a simple count of vehicles in each category, and the number of persons in each vehicle. Traffic in the direction of heavier movement only was counted and the field data were summarized for each signal cycle (80 seconds). From this study a comparison was made between the space occupied by streetcars and automobiles, and of the load carried by vehicles of each type.

Average Load Carried

A summation of data for the heaviest hour at each of the ten study sites shows that 690 streetcars and 9,126 automobiles were observed. The 690 streetcars transported 35,070 passengers while the automobiles carried a total of 17,932 passengers, including drivers. Average carried loads were 50.83 persons for streetcars as against 1.97 for automobiles. The average streetcar carried 25.8 times as many people as the average automobile.

Space per Vehicle

The space occupied by the average vehicle of each type may be measured by dividing the number of vehicles per hour by the width of street available to that type of vehicle.

At the average site, 4.7 streetcars per hour per foot of width availed themselves of the space allotted to them, that is, the width of the car-track lane to the center of the street and including the loading platform. The average number of automobiles utilizing the space between the curb and platform was 35.9 per hour per foot of width. For operating conditions as they occurred during the week of the study, the average streetcar occupied 7.6 times as much space in the traffic stream as the average automobile occupied.

Travel Time

For the purpose of measuring streetcar travel time, three streets were selected and these were either within or adjacent to the central business district of Washington.

Travel time for streetcars was determined by observers working in pairs on the three streetcar lines. The observers were stationed several blocks apart and each recorded the exact time that every streetcar passed his station, together with the identification number on the side of the car. Later, the records for the two observers were compared and the elapsed time for each streetcar to traverse the known distance between the observation points was computed. Data were collected for 750 streetcars covering approximately 450 vehicle-miles of travel. The average travel time during periods of peak traffic was found to be 11.0 minutes per mile.

Automobile Operation on Routes Followed by Streetcars

The method used, as well as the results obtained, in determining the average number of people in automobiles has already been described. The average occupancy of automobiles was 1.97. The rate of travel for automobiles as measured in the study of bus operation (Table 4) is used in comparing automobile and streetcar operation.

Relative Efficiencies of Automobiles and Streetcars

The procedure for calculating the relative efficiencies of automobiles and streetcars is the same as that followed in comparing automobiles and buses. Relative efficiencies

TABLE 12

RELATIVE EFFICIENCIES OF AUTOMOBILES AND STREETCARS IN THE UTILIZATION OF STREET SPACE AND MOVEMENT OF PEOPLE IN A DOWNTOWN AREA DURING PEAK HOURS

Type of Vehicle	Travel Time (min per mi)	Carried Load (persons per veh)	Space per Vehicle (auto. units)	Relative Efficiency
Automobiles	6.0	2.0	1.0	1.0
Streetcars	11.0	50.8	7.6	1.8

of streetcars, as compared with automobiles, together with values of the variables used in developing these efficiencies, are shown in Table 12. The streetcar is shown as being 1.8 times as efficient as the automobile in downtown Washington. Data were not obtained for other areas of the city.

SUMMARY

The operation of each of three types of mass transit vehicles has been compared with automobiles that were using the same streets during approximately the same period of time. The three variables used in the comparison were travel time, number of passengers, and space occupied in the traffic stream. The resulting efficiencies, based on a comparison between these variables, are summarized in Table 13.

Figures in Table 13 cannot be used to compare transit vehicles in one city with transit vehicles in another city because the automobiles, which were used as a standard for comparison, varied in their operation on different types of streets, and more particu-

TABLE 13

RELATIVE EFFICIENCIES OF VARIOUS TYPES OF VEHICLES IN THE UTILIZATION OF STREET SPACE AND MOVEMENT OF PEOPLE

Type of Vehicle	Relative Efficiency		
	Downtown	Inter-mediate	Outlying
Automobile	1.0	1.0	1.0
Bus (surface street)	3.7	4.6	2.6
Bus (freeway)	-	7.2	-
Trolley coach	6.3	8.7	6.3
Streetcar	1.8	-	-

TABLE 14

SPACE IN THE TRAFFIC STREAM OCCUPIED BY ONE PERSON IN VEHICLES OF DIFFERENT TYPES

Type of Vehicle	Relative Amount of Space Occupied		
	Downtown	Inter-mediate	Outlying
Automobile	1.00	1.00	1.00
Streetcar	0.30	-	-
Bus (surface street)	0.17	0.13	0.21
Bus (freeway)	-	0.10	-

TABLE 15

RELATIVE EFFICIENCIES OF VEHICLES OF VARIOUS TYPES IN THE UTILIZATION OF STREET SPACE, Considering Space Occupied per Vehicle and Number of Persons per Vehicle

Type of Vehicle	Relative Efficiency of Utilization of Street Space		
	Downtown	Inter-mediate	Outlying
Automobile	1.00	1.00	1.00
Streetcar	3.38	-	-
Bus (surface street)	5.95	7.60	4.67
Bus (freeway)	-	10.00	-

larly between cities. Automobiles in Atlanta, for example, traveled slower and carried fewer passengers than did automobiles in Washington. It is primarily for this reason that trolley coaches in Atlanta are shown to have a higher efficiency than buses in Washington. The inference should not be drawn that trolley coaches would be almost twice as efficient in Washington as buses are. In the case of streetcars vs buses such a comparison is valid because both of these transit vehicles are compared to automobiles operating over the same Washington streets.

As another precaution, Table 13 should not be interpreted as meaning that automobiles could be substituted for transit vehicles in the numbers shown in the table and the same total number of persons be transported by automobile alone as are presently being moved by transit vehicles and automobiles combined.

Fewer people could be transported in automobiles alone than can be moved on a street by automobiles in combination with transit vehicles, but this smaller number of people would reach their destinations in a shorter length of time. If the concern is with absolute numbers of persons that can be moved on a street without regard to rate of travel, then the transit vehicle enjoys a much greater advantage than is reflected in Table 13. The space in the traffic stream which a person occupies while traveling in various types of vehicles is a more reliable measure of the number of people that can be moved past a point in vehicles of each type. Table 14 shows the relative amount of space in the traffic stream a person occupies while traveling in each of the several types of vehicles.

Translated into relative efficiencies of street-space utilization the reciprocals of the figures in Table 14 are shown in Table 15. As in the case of the figures in Table 13, a direct comparison cannot be made between trolley coaches and buses or streetcars because trolley coach operation is related to automobile operation in Atlanta, whereas bus and streetcar operation is related to automobile operation in Washington.

CONCLUSIONS

The investigations here reported upon are exploratory in nature and the interpretations placed upon the results are based on operating conditions as they actually occurred during the periods studied. Operating conditions are subject to change and the rates at which people are moved in various types of vehicles do not in any sense represent the maximum number of people that could be moved under hypothetical operating conditions. For example, in no instance was the potential capacity of the car-track lane fully utilized by mass transit vehicles although the routes studied were the most heavily traveled in the city; one site, with 113 streetcars in one hour, approached what is considered the limit in numbers that can use a street in one direction. More people could have been moved by streetcars without any further encroachment upon the street space, but this could happen only if more people wished to avail themselves of this mode of transportation, and if more streetcars were made available for their use.

In the case of automobiles, the potential capacity of the street space was more fully utilized by vehicles at most of the study sites than was the case for streetcars. Automobiles were only one-third filled to their capacity, however. To hypothesize that more streetcars could operate over a single track (if there were people to fill them) merely invites a parallel hypothesis that more people could band together as group riders in automobiles if they should so choose. For current operations, comparisons as made in this report would seem to be founded on as firm a basis as any that might be conceived. On this premise the following conclusions are drawn for peak-hour operations:

1. In Washington, D.C., buses were 3.7 times as efficient as automobiles on downtown streets, 4.6 times as efficient in intermediate areas, and 2.6 times as efficient as automobiles in outlying areas. On a freeway, buses in Atlanta, Georgia, were 7.2 times as efficient as automobiles.

2. In Atlanta, trolley coaches were 6.3 times as efficient as automobiles on downtown streets, 8.7 times as efficient in intermediate areas of the city, and 6.3 times as efficient as automobiles in outlying areas.

3. In Washington, streetcars were 1.8 times as efficient as automobiles on downtown streets.

4. The efficiencies of mass transit vehicles in Washington, as derived in this investigation, cannot be compared directly with those in Atlanta because of differences in the operation of automobiles in the two cities. Automobiles in Washington traveled faster and carried more passengers than automobiles in Atlanta.

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This investigation is a product of the joint efforts of several agencies. The studies of travel time and of passengers carried by trolley coaches and automobiles in Atlanta were performed by personnel of the State Highway Department of Georgia, with the cooperation of the Atlanta Transit Company in the selection of routes and the furnishing of riders' passes to those engaged in the study. Field work for a determination of the space occupied by trolley coaches in the traffic stream was performed by the Traffic Engineering Department of the City of Atlanta.

Field work for the bus study in Washington was performed by Bureau of Public Roads personnel assisted by personnel furnished by the Capital Transit Company. Participation by that company included the services of five observers under the supervision of the Research and Planning Department. The company also participated in establishing procedures for the study and in furnishing riders' passes to all engaged in the study. The W.V. and M. Coach Company of Arlington, Virginia, supported the study furnishing free fares to observers operating on its lines.

New Methods of Capacity Determination for Rural Roads in Mountainous Terrain

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● 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 construction, reconstruction, and improvements needed to bring the existing systems up to these performance levels. The levels of performance which are established must be feasible and based on the current desires of traffic to move safely and efficiently, with due consideration being given to the future demands of highway transportation since it is such an important factor in the national economy.

Performance levels may be measured, and also specified, in terms of safe operating speeds. Comprehensive studies as well as past practices have shown that drivers demand, and that it is more feasible to construct, facilities that will permit higher operating speeds in level terrain than in rough or mountainous terrain, on primary highways carrying most of the long-distance travel than on local roads where the average trip length is shorter, and for roads on the same system carrying the higher traffic volumes than for those carrying the lower traffic volumes. Although 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 maximum 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] In short, it is the maximum safe 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 items 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 higher design speeds.

The operating speed is the highest over-all 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 design speed, traffic volume, and 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 perform passing maneuvers safely. 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 of a highway.

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 the construction of a new highway.

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[1] AASHO definition.

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 carrying more than 1,800 vehicles per day are shown by Table 1.[2]

After these tolerable conditions and standards in terms of service to traffic had been agreed upon, it was relatively easy to establish the design requirements for new construction (Table 2A) from the information now contained in the AASHO policy on "Geometric Design of Rural Highways" and to prepare Table 3 from the results of traffic operation and capacity studies conducted during the past several years.

Table 3 shows the average daily traffic volumes that can be accommodated by a two-lane highway constructed to a given design speed with various percentages of the highway having slight distances in excess of 1,500 to 1,000 ft. These values are for the average conditions applicable in West Virginia which are:

1. The 30th highest hourly volume during the year is 12 percent of average daily traffic for that year.

TABLE 1

TOLERABLE CONDITIONS FOR EXISTING RURAL HIGHWAYS CARRYING OVER 1,800 VEHICLES PER DAY AND STANDARDS FOR NEW CONSTRUCTION OR RECONSTRUCTION IN TERMS OF THE SERVICE PROVIDED

Highway System	Terrain	Tolerable Conditions		Construction Standards	
		Operating Speed, mph	Design Speed, mph	Operating Speed, mph	Design Speed, mph
Interstate	Valley or level	45-50	60	50-55	70
	Rolling	40-45	50	45-50	60
	Mountainous	40-45	45	45-50	60
Other	Valley or level	45-50	60	50-55	70
	Rolling	40-45	50	45-50	60
	Mountainous	35-40	40	40-45	60

2. During the 30th highest hourly volume of a year, trucks with dual tires are 5 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 alignments 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 different highways constructed to modern standards may provide radically different operating conditions, even though their traffic volumes and their design

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

TABLE 2A

GEOMETRIC STANDARDS FOR NEW CONSTRUCTION FOR RURAL PRIMARY STATE HIGHWAYS

	TWO-LANE HIGHWAYS — FUTURE AVERAGE DAILY TRAFFIC VOLUME GROUPS: LESS THAN 500			500 TO 1800			1800 TO 3000			OVER 3000			INTERSTATE SYSTEM			FOUR-LANE HIGHWAYS		
TERRAIN	VALLEY	ROLLING	MOUNTAINOUS	VALLEY	ROLLING	MOUNTAINOUS	VALLEY	ROLLING	MOUNTAINOUS	VALLEY	ROLLING	MOUNTAINOUS	VALLEY	ROLLING	MOUNTAINOUS	VALLEY	ROLLING	MOUNTAINOUS
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 (MPH)	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	475	350	275	700	525	400	700	525	400	700	525	525	700	525	525
LANE WIDTH (Feet)	10			11 1/			12			12			12			12 2/		
SHOULDER WIDTH (Feet) — IN CUT	5	3	3	6	4	4	8	6	4	8	8	4	8	8	4	8	8	4
SHOULDER WIDTH (Feet) — ON FILL	7	5	5	8	8	7	10	8	8	10	10	8	10	10	8	10	10	8
PASSING SIGHT DISTANCE (% Available) — 1500 Ft.	PERCENT FEASIBLE			10	5	—	10	5	—	10	5	—	10	5	—	NOT APPLICABLE		
PASSING SIGHT DISTANCE (% Available) — 1000 Ft.	PERCENT FEASIBLE			—	—	5	—	—	5 OR AS REQUIRED FOR CAPACITY	—	—	5	—	—	5			
GRADE (Percent)	5	7	9	4	5	7	7% MAXIMUM UNLESS PERCENT AND LENGTH OF GRADE IS DETERMINED BY CAPACITY CALCULATION									7		
	ADD 2% FOR GRADES UNDER 750 FT LONG			ADD 1% FOR GRADES UNDER 750 FT. LONG														
RIGHT OF WAY WIDTH 3/ (Feet)	60			80			100			120			120			200		
SURFACE TYPE	SURFACE TREATMENT			MEDIUM			HIGH			HIGH			HIGH			HIGH		
BRIDGES — LOADING	H15-S12 OR ONE H20 TRUCK, WHICHEVER PRODUCES THE GREATER STRESS									H20-S16 FOR INTERSTATE ROUTES AND OTHER PRIMARY ROUTES CARRYING HEAVY TRUCK TRAFFIC COMPARABLE TO INTERSTATE ROUTES								
BRIDGES — WIDTH	UNDER 50 FT. LONG — FULL ROADWAY WIDTH; OVER 50 FT LONG — APPROACH PAVEMENT PLUS 4 FT									UNDER 50 FT. LONG — FULL ROADWAY WIDTH; OVER 50 FT LONG — APPROACH PAVEMENT PLUS 4 FT AND MEDIAN ON FOUR-LANE HIGHWAYS								
BRIDGES — VERTICAL CLEARANCE	NOT LESS THAN 14.5 FEET																	

1/ ON NEW CONSTRUCTION OR RECONSTRUCTION 12 FT. LANES SHOULD BE USED WITH VOLUMES ABOVE 1200 PER DAY WHEN TRUCK TRAFFIC IS MORE THAN 5 PERCENT.

2/ ALL FOUR-LANE HIGHWAYS SHALL BE DIVIDED WHENEVER POSSIBLE THE MEDIAN SHALL BE AT LEAST 20 FT. WIDE, AND IN NO CASE LESS THAN THE 4-FOOT BARRIER TYPE.

3/ CONTROL OF ACCESS SHALL BE PROVIDED ON ALL NEW LOCATIONS ON THE INTERSTATE HIGHWAY SYSTEM AND ON OTHER PRIMARY HIGHWAYS CARRYING LARGE VOLUMES OF TRAFFIC

TABLE 2B

TOLERABLE CONDITIONS FOR RURAL PRIMARY STATE HIGHWAYS

		TWO-LANE HIGHWAYS															FOUR-LANE HIGHWAYS		
		AVERAGE DAILY TRAFFIC VOLUME GROUPS																	
		LESS THAN 500			500 TO 1800			1800 TO 3000			OVER 3000			INTERSTATE SYSTEM					
TERRAIN		VALLEY	ROLLING	MOUNTAINOUS	VALLEY	ROLLING	MOUNTAINOUS	VALLEY	ROLLING	MOUNTAINOUS	VALLEY	ROLLING	MOUNTAINOUS	VALLEY	ROLLING	MOUNTAINOUS	VALLEY	ROLLING	MOUNTAINOUS
DESIGN SPEED (M PH)		40	35	30	50	40	35	60	50	40	60	50	40	60	50	45	60	50	45
OPERATING SPEED (MPH)		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 (Feet)		275	225	200	350	275	225	475	350	275	475	350	275	475	350	300	475	350	300
LANE WIDTH (Feet)		9			10	10[1/]		10			10			10			10		
SHOULDER WIDTH (Feet)	IN CUT	3	3	2	6	3	3	6	4	4	6	4	4	6	4	4	6	4	4
	ON FILL	3	3	3	6	4	4	8	6	4	8	6	4	8	6	4	8	6	4
PASSING SIGHT DISTANCE (% Available)	1500 Ft	NOT REQUIRED			NOT REQUIRED			10	5	—	10	5	—	10	5	—	NOT APPLICABLE		
	1000 Ft	NOT REQUIRED			NOT REQUIRED			—	—	5	—	—	5	—	—	5			
								OR AS REQUIRED FOR CAPACITY											
GRADE (Percent)		7	8	10	5	7	8	8	8	8	7	7	7	7	7	7	7		
		ADD 1% FOR GRADES UNDER 1000 FT LONG			ADD 1% FOR GRADES UNDER 750 FT LONG			MAXIMUM UNLESS PERCENT AND LENGTH OF GRADE IS DETERMINED BY CAPACITY CALCULATION											
SURFACE TYPE		SURFACE TREATMENT			SURFACE TREATMENT[2/]			MEDIUM[2/]			MEDIUM[2/]			MEDIUM[2/]			MEDIUM[2/]		
BRIDGES	LOADING	H15 FOR ALL BRIDGES ON THE PRIMARY SYSTEM																	
	WIDTH	NOT LESS THAN WIDTH OF APPROACH SURFACE			NOT LESS THAN WIDTH OF APPROACH PAVEMENT PLUS 2 FEET														
	VERTICAL CLEARANCE	NOT LESS THAN 13 FEET																	

1/ LANE WIDTHS OF 9 FT. MAY BE ACCEPTED AS TOLERABLE UNDER 800 VEHICLES PER DAY WITH A SMALL PERCENTAGE OF TRUCKS.

2/ GOOD SURFACE CONDITION REQUIRED

CONTROL OF ACCESS NOT REQUIRED

speeds are identical. Average speeds, for example, will be much higher on a highway with few 5-degree curves and considerable tangent alignment than on a highway with many 5-degree curves and little tangent alignment. 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 alignment will accommodate higher volumes of traffic than a similar highway with many curves of the same degree and less tangent alignment. In relating the operating speed of a highway to its capacity, therefore it is necessary to determine the "average highway speed," especially for existing highways.

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 alignment and profile to capacities. It is an approach, however, which must be employed to obtain reasonable accuracy in capacity determinations, especially for existing highways.

TABLE 3

CONSTRUCTION STANDARDS FOR CAPACITIES OF 2-LANE HIGHWAYS, 12-FOOT LANES

(Based on 5 percent trucks during 30th highest hour factor of 12 percent ADT)

Percentage of Highway with Passing Sight Distance[a]		Average Daily Traffic Volume			
				Mountainous	
		Valley or Flat	Rolling	Interstate	Other
1,500 ft	1,000 ft	Oper. Speed 50-55 Design Speed 70	Oper. Speed 45-50 Design Speed 60	Oper. Speed 45-50 Design Speed 60	Oper. Speed 40-45 Design Speed 60
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

[a] Percentage of 1,500-ft passing sight distance is used for all operating speeds except those below 45 mph. The 1,000-ft values are applicable to all operating speeds.

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 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. The lower the design speed, the greater is the likelihood of the occurrence of such sections. Invariably there are sections where the designer utilizes values that are adequate for higher speeds than the design speed assumed. 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 speed on a 2-lane highway varies with the

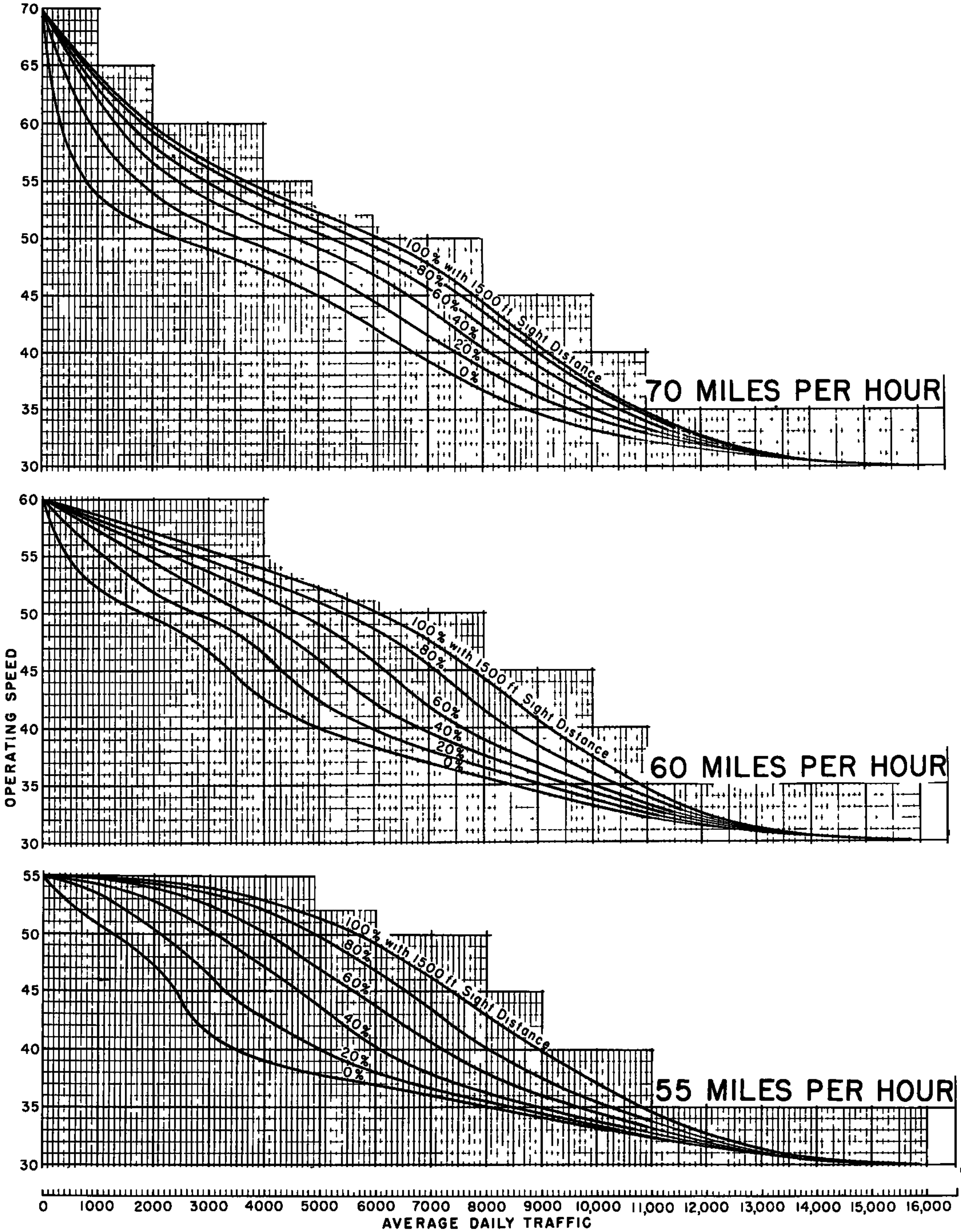

Figure 1. Effect of traffic volume and available passing sight distance of two-lane roads on operating speed for various average highway speeds. (Computed on basis of no grades exceeding 3 percent, 12-ft lanes, 12 percent design hour, 5 percent dual-tired commercial vehicles in the design hour, and a truck equivalent of 2).

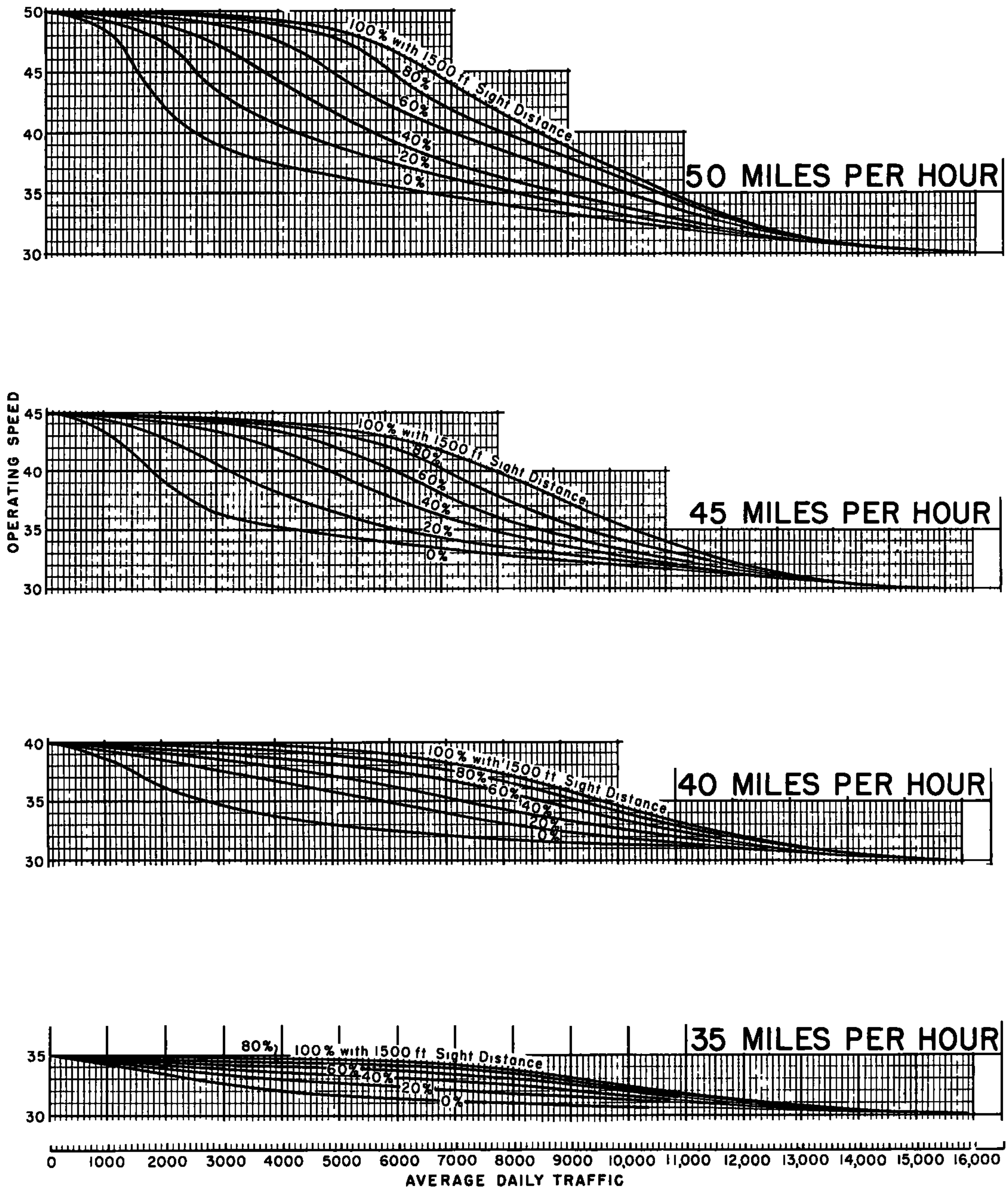

Figure 2. Effect of traffic volume and available passing sight distance of two-lane roads on operating speed for various average highway speeds. (Computed on basis of no grades exceeding 3 percent, 12-ft lanes, 12 percent design hour, 5 percent dual-tired commercial vehicles in the design hour, and a truck equivalent of 2).

average highway speed, the percentage of highway having 1,500-ft passing sight distance, and the traffic volume. The average daily traffic volumes in these charts are for (a) terrain which is essentially level, (b) 12-ft traffic lanes, (c) 5 percent dual-tired vehicles with a passenger car equivalent of 2, and (d) a 30th highest hourly volume during the year of 12 percent of the average daily traffic. They were prepared for the Tennessee programing study from the information contained in Table 4, which was prepared for the 1953-54 West Virginia needs study. Table 4, in turn, was prepared

TABLE 4

AVERAGE DAILY CAPACITIES OF 2-LANE HIGHWAYS

(Level terrain, 5 percent dual-tired vehicles, truck factor 2.0[a]
30th highest hour factor of 12 percent ADT)

Operating Speed, mph	Percentage of Highway with Passing Sight Distance of		Average Daily Traffic at Average Highway Speed of					
	1,500 ft	800 to 1,000 ft	70 mph	60 mph	55 mph	50 mph	45 mph	40 mph
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

[a]For West Virginia; normally 2.5.

from the results of extensive highway capacity studies conducted by the Bureau of Public Roads in cooperation with the various state highway departments and include the results reported in the "Highway Capacity Manual," supplemented by more recent investigations.

Figures 1 and 2 contain curves representing roadways with sight distances that are continuously in excess of 1,500 ft to those that have no 1,500-ft sight distances. 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-ft sight distance is fairly evenly distributed between the limits of 1,500 ft 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 2-lane highways during capacity volumes represent principally level tangent sections well removed from sharp horizontal or vertical curves. For this reason, all curves except the ones for 100 percent of 1,500-ft sight distance are shown as light lines above 11,000 vehicles per day. There is still considerable question as to whether all the curves for the same average highway speed meet at a common point on the right, or whether the possible capacity and the speed at this capacity are slightly lower for the highways with the poorer alignment than for those with a continuous sight distance in excess of 1,500 ft. This, however, is not too important a consideration because the practical capacities of 2-lane highways a well within the range for which reliable data are available.

The charts may be used either to determine the operating speed for a given traffic volume or the traffic volume which the highway will accommodate at a given operating speed. When it is desired to determine the capacity at a given operating speed for lan widths other than 12 ft, for 30th highest hourly factors other than 12 percent, for truck percentages other than 5 percent, or for truck equivalents other than 2, the following factors must be applied to adjust the capacity volumes to the prevailing or estimated future conditions:

1. For 11-ft lanes multiply the volumes by 0.86; for 10-ft lanes, by 0.77.
2. When the 30th highest hour factor is other than 12 percent, multiply the volumes by $\frac{12}{\text{actual percentage}}$.
3. When there is other than 5 percent trucks during the peak hour or the truck equiv alent is greater than 2, as it will be on grades and in rolling or mountainous terrain, multiply the volumes by $\frac{105}{100 - P + PT}$, where P is the percentage of trucks and T is the truck equivalent in terms of passenger cars.

The operating speed for a given traffic volume when conditions other than those used for these charts are applicable may be determined by employing the reciprocal of these correction factors to the given traffic volume before entering the chart.

The Appendix contains eight tables for the conditions most prevalent on 2-lane roads in West Virginia. The number of charts or tables that can be prepared for other combi nations of the many variable conditions is almost unlimited. A similar set of tables ma be prepared for the conditions prevailing within any state or area.

FOUR-LANE DIVIDED HIGHWAYS

Figure 3 shows the relation between operating speeds, average highway speeds, and traffic volumes on 4-lane divided rural highways free from the influence of intersection The lowest curve represents the minimum speed at which traffic must flow to attain a

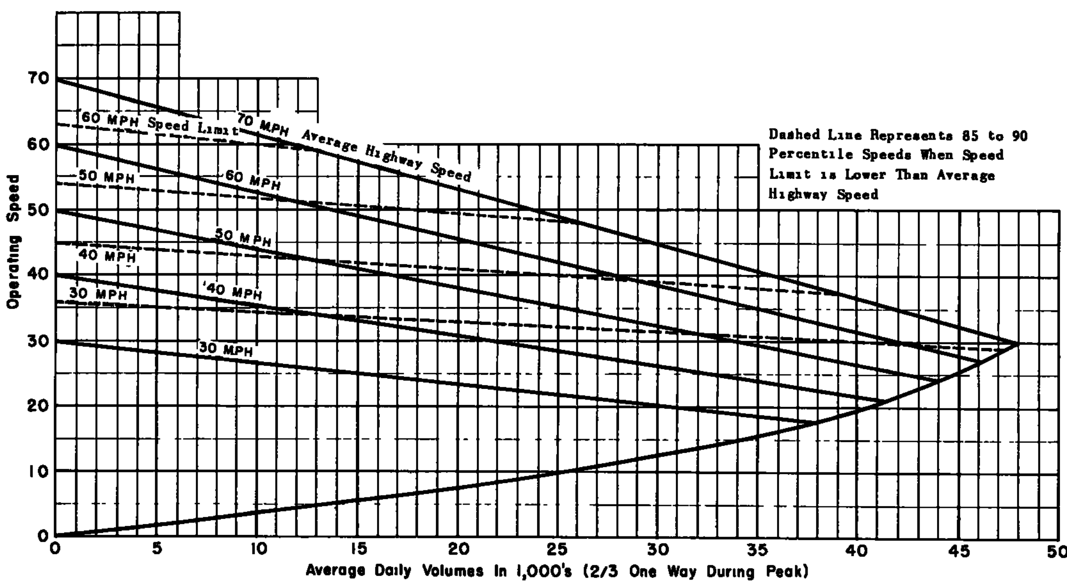

Figure 3. Operating speeds on 4-lane highways for various average highway speeds, in direction of heavier travel.

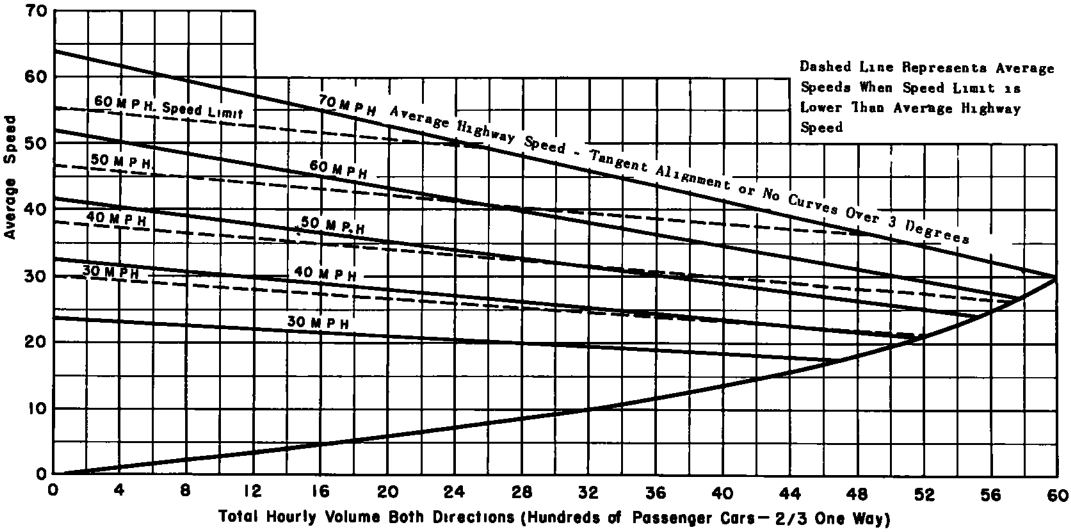

Figure 4. Average speed on 4-lane highways for various average highway speeds, in direction of heavier travel.

given traffic volume. For example, traffic must be traveling at least 10 mph for a 4-lane highway to accommodate the 30th highest hourly volume when the average daily traffic is 25,000 vehicles.

The other solid lines of Figure 3 represent the normal operating speeds during various traffic volumes for different average highway speeds. Any point representing the speed-volume relationship must fall between the lower curve and the line representing the average highway speed.

The dashed lines show the effect of an enforced speed limit on the speed-volume relationship. A speed limit has an effect on the operating speed only when it is lower than the highway speed. Also, it has an effect only when the traffic volume is below that at which the dashed speed limit line intersects the solid line corresponding to the highway speed. At higher volumes, the solid lines show the normal speed-volume relationship, because at these volumes the speeds are governed by the traffic density rather than by the speed limits.

Figure 4 is similar to Figure 3 except that the average speed, rather than the operating speed, is related to the traffic volume. Figure 3 also shows the daily volumes based on a 30th highest hour factor of 12 percent and includes 5 percent trucks with a passenger car equivalent of two, whereas Figure 4 shows hourly volumes and includes no trucks.

These charts represent average conditions found on modern highways throughout the United States. In some areas, such as the central states where the terrain is level and speeds are higher than for the country as a whole, the speeds as shown by these charts will be somewhat low, especially for the low traffic volumes. For certain other areas they may be high, but in general any difference will not be great and the relative speeds for the different conditions will be accurate.

The traffic volumes or capacities at a given operating speed or at a given average speed are shown in terms of passenger cars in two 12-ft lanes for the one direction of travel. Daily and hourly volumes, or capacities for various percentages of trucks and a range of truck factors, may be determined by standard procedures.

The results for multilane highways, as shown by Figures 3 and 4, explain to a large extent the many variations in the speed volume relationship found by other investigators. Sometimes they have found that an increase in the traffic volume or density results in only a very slight or no drop in speeds. This would be the case, as shown by the dashed lines of Figures 3 and 4, when a speed limit or factors other than the traffic density are exerting a controlling influence on vehicle speeds.

The results of still other investigators show a curvilinear relationship, with the speeds dropping at an increasing rate as the traffic density increases. This would occur as the traffic volumes exceeded the range within which the speed limits were effective and especially when the volumes approached possible capacities. At volumes approaching possible capacities on multilane facilities (above 1,500 vehicles per lane), the safety factor for capacity, as indicated by the distance between the upper and lower curves of Figures 3 and 4, decreases rapidly, with the result that a slow driver or some other minor condition interrupting the normal flow of traffic can cause a sudden slowdown of all vehicles, with speeds decreasing from a point on one of the higher curves of Figures 3 and 4 to a point on the bottom curve, or to any intermediate point. The closer the possible capacity is approached, the greater is the possibility of such an occurrence.

The most baffling results obtained from speed-volume investigations are those which show an increase in speed with an increase in volume. Generally this occurs when a study is started during off-peak hours with light traffic and is continued through the peak or rush-hour volumes in the afternoon. As the traffic volume increases, the percentage of repeat drivers in a hurry to get home increases, with the result that speeds show little or no decline and oftentimes increase temporarily with the traffic volume. When capacity volumes are reached or closely approached, there is an abnormal decrease in speeds, producing the curvilinear relation between speed and traffic volume. Studies of this type do not show the true effect of increased volume or speeds, because 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 by 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. Type of terrain through which the highway is located.
2. Average highway speed and frequency of occurrence of sharp curves that cause abnormally low speeds.
3. Percentage of the highway on which the passing sight distance exceeds 1,500 ft. On highways for which an operating speed of 40 mph or less has been specified, the percentage of highway with an 800- to 1,000-ft sight distance is required whenever there is a low percentage of the 1,500-ft 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.

Terrain

Generally the alignment 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. Nevertheless, the fact that the existing highway has many sharp curves and steep grades does not necessarily mean that a much better alignment 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 a terrain, however, 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-mph highway speed, even though the test car was not necessarily operated at this speed. Such sections are, however, comparatively rare in West Virginia.

This method of determining the average highway speed and of 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 mph) to determine the length and location of all sections with sight

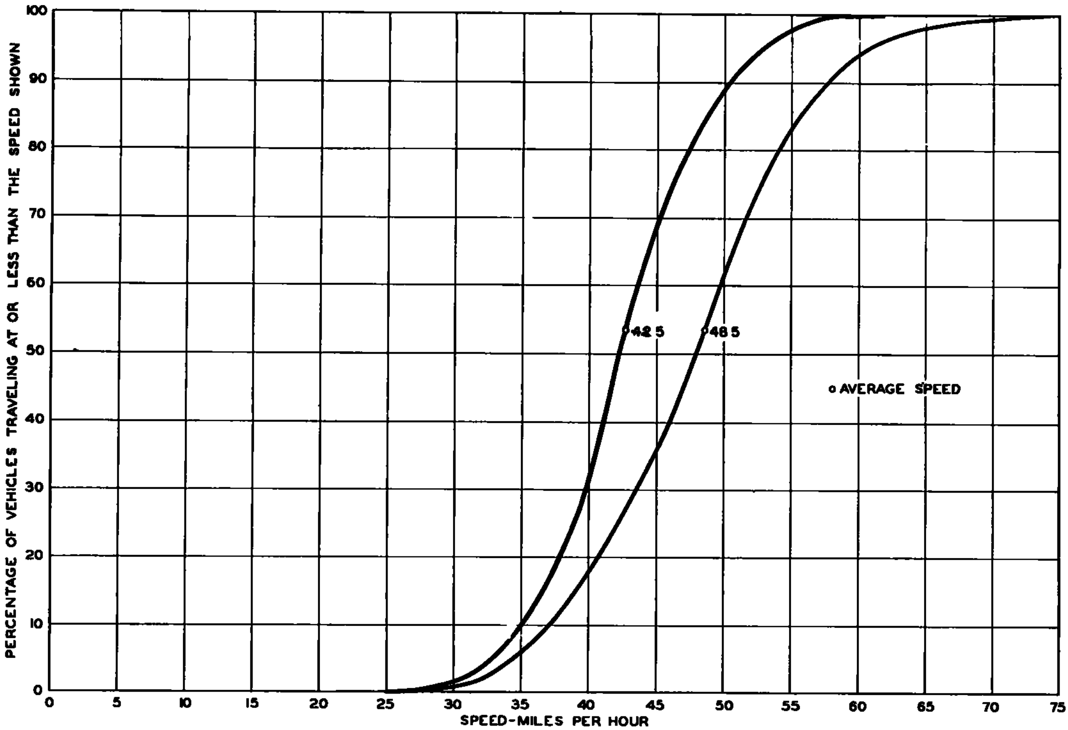

Figure 5. Distribution of normal passenger car speeds used for truck factors.

distances in excess of 1,000 ft and 1,500 ft, in lieu of more accurate and detailed sight distance information. The driver informed the passenger, who acted as the recorder, each time that there was a change in the sight distance from some value below 1,000 ft or 1,500 ft to a value above 1,000 or 1,500 ft. He also informed the recorder each time that 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 ft and the percentage in excess of 1,500 ft.

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

TABLE 5

TRUCK FACTOR FOR VARIOUS TRUCK SPEEDS AS RELATED TO NORMAL PASSENGER CAR SPEEDS

Truck Speed, mph	Truck Factor, pass. car equiv.		
	For Average Passenger Car Speed of 47.5 mph[a]	For Average Passenger Car Speed of 42.5 mph[b]	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

[a]Distribution as shown by curve A of Figure 3.

[b]Distribution as shown by curve B of Figure 3.

dual-tired vehicle is equivalent, in a capacity sense, to 2 passenger cars on multilane highways and to 2.5 passenger cars on 2-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. (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.)

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

The results of these studies have shown that the 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 5 shows how the truck factor varies with the truck speed for two different passenger car speed distributions, as shown by Figure 5. The higher the passenger car speeds, the higher are the truck equivalents. The factors in the right-hand column 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

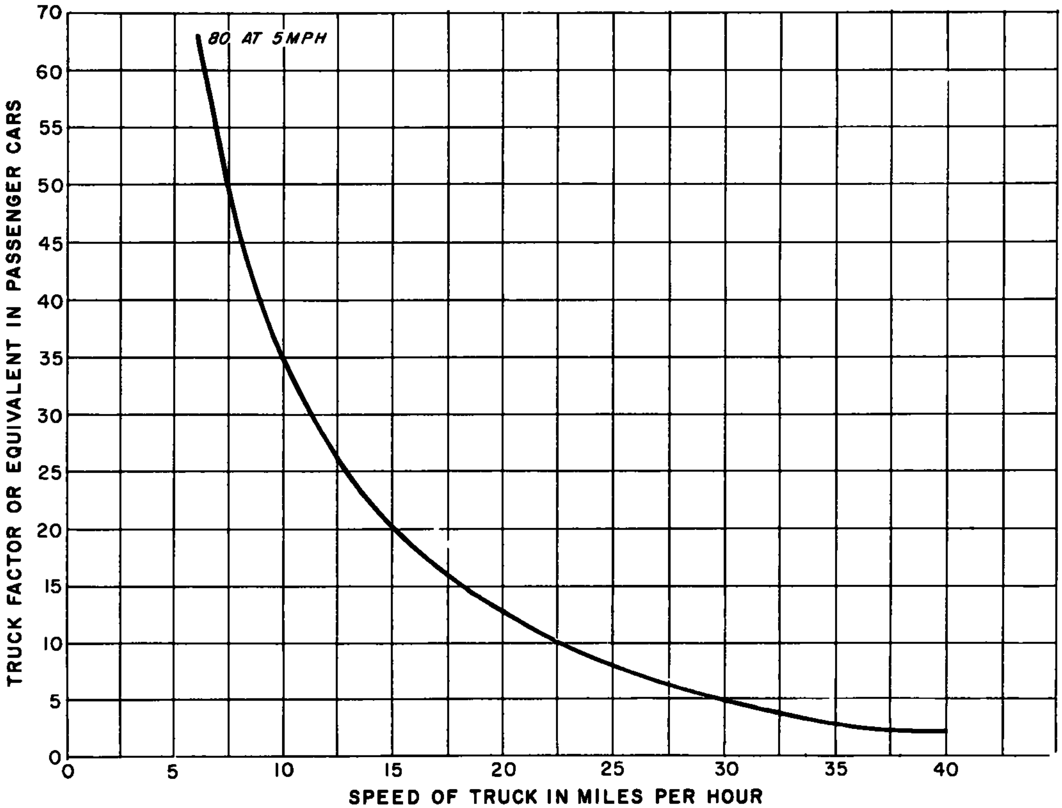

Figure 6. Truck factor for various average truck speeds.

knowing the average speed for all trucks if the passenger car speeds are within the limits of those shown by 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 highway system. In mountainous terrain, however, this approaches an impossible task. This is especially true for West Virginia. Therefore, a unique method was employed to obtain the average truck factor for each section of highway and for each grade or combination of grades on all roads in West Virginia likely to carry more than 1,800 vehicles per day during the next 20 years. The method involved the selection of a typical truck with a typical load. This truck was driven over the highway system at its maximum safe speed consistent with normal truck operation to obtain a continuous speed profile. The speed of the truck and its odometer reading were recorded at the bottom and top of each grade, at crossroads or other control points, each time the gears were shifted, and each time there was a change of 5 mph in the speed of the truck. When the truck reached a crawl speed on long grades, the crawl speed was recorded to the nearest mile per hour. The truck was operated in both directions on the more important roads to get a speed profile for each direction of travel.

The control truck and its load were selected to obtain a weight-power ratio of 325 lb 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 lb, which was 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. Because Figure 6 is not a straight line, the possible speed of the control truck on an upgrade was purposely 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 mph is 11.5, or $(\frac{3+20}{2})$. A truck factor of 11.5 is represented by a speed of 21 mph rather than 25 mph (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 studies of trucks on grades obtained at spot locations and also over the entire length of long grades by stopwatch studies showed that the average truck factor obtained by this procedure was somewhat lower than the truck factor obtained by using the speed of the control truck. The difference varied from 10 to 20 percent. Inasmuch as 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 presentday commercial vehicles as found in West Virginia, the load on the control truck should have been reduced about 5,000 lb.

The average speeds of the control truck on 3 to 7 percent uniform grades up to 6 miles long are shown by Figure 7 and Table 6. 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 mph.

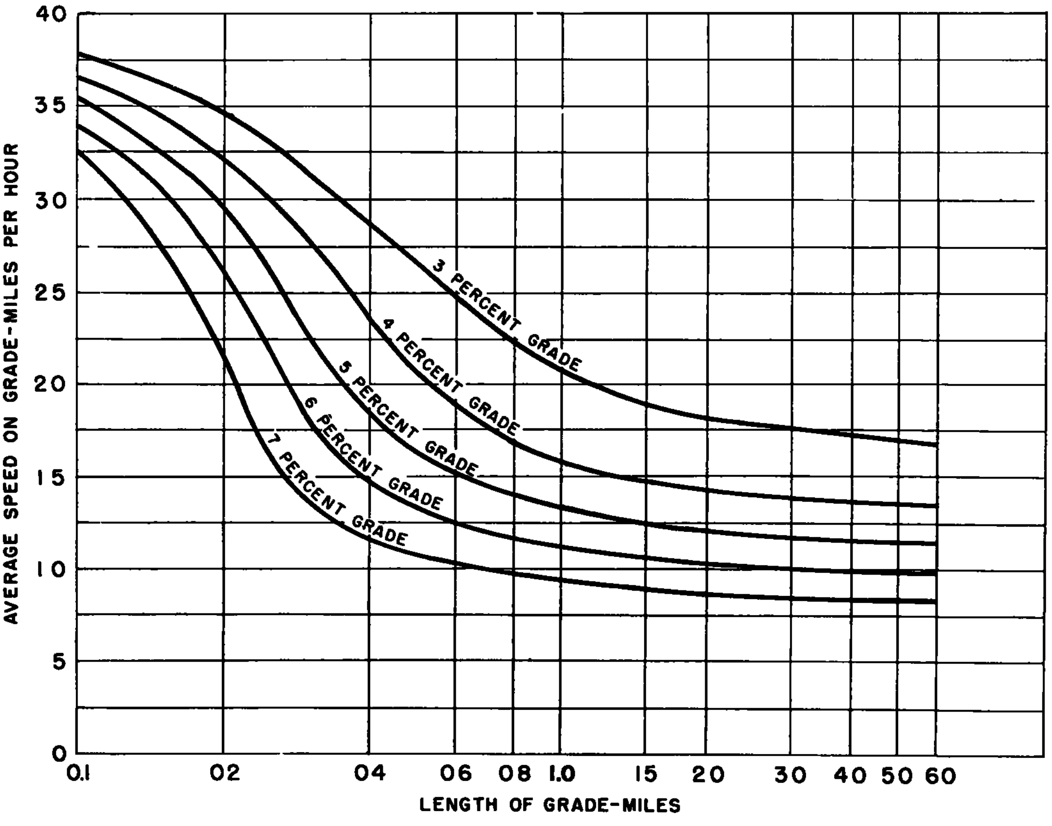

Figure 7. Average speed of control truck on grades.

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 is 40 mph (initial distance 85), the speed at the top of a 4 percent grade 1,000 ft long will be 26 mph (final distance 1,085). Similarly, if this same grade is approached at a speed of 30 mph, the speed at the top will be 17 mph.

The dashed curves emanating from 9 mph show the maximum performance of vehicles when the approach speed is so low that the vehicle must accelerate to eventually

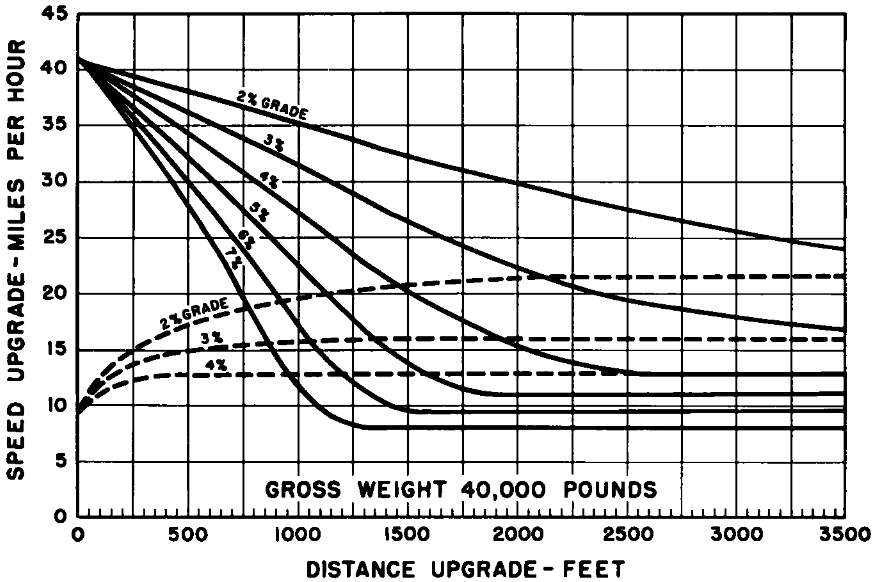

Figure 8. Effect of length of grade on the speed of medium motor vehicles.

TABLE 6

AVERAGE SPEED OF TYPICAL TRUCK ON GRADES, ENTERING SPEED 40 MPH

Length of Grade, mi	Average Speed, mph				
	3% Grade	4% Grade	5% Grade	6% Grade	7% Grade
0.1	37.3	36.1	35.2	34.0	32.6
0.2	34.6	31.7	29.3	25.8	21.4
0.4	28.4	23.4	18.2	14.5	11.8
0.6	24.6	18.5	14.9	12.4	10.2
0.8	21.9	16.6	13.7	11.5	9.5
1.0	20.4	15.7	13.1	11.0	9.2
1.5	18.7	14.6	12.3	10.5	8.8
2.0	17.9	14.1	11.9	10.2	8.5
3.0	17.3	13.6	11.6	10.0	8.4
4.0	16.9	13.4	11.5	9.8	8.3
5.0	16.7	13.3	11.4	9.8	8.2
6.0	16.6	13.2	11.3	9.7	8.2
Sustained Speed	16.0	12.8	11.0	9.5	8.0
Distance to reach sustained speed, mi	0.78	0.60	0.37	0.28	0.24

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 mph to the sustained speed of 21.5 mph, an increase of only 1.5 mph, the vehicle would have to travel 1,050 ft.

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 (1, 2, 3, 4, 5, 6). This was not necessary for the West Virginia needs study because the truck was operated over all routes under consideration.

If the grades had been uniform and their lengths and gradients 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

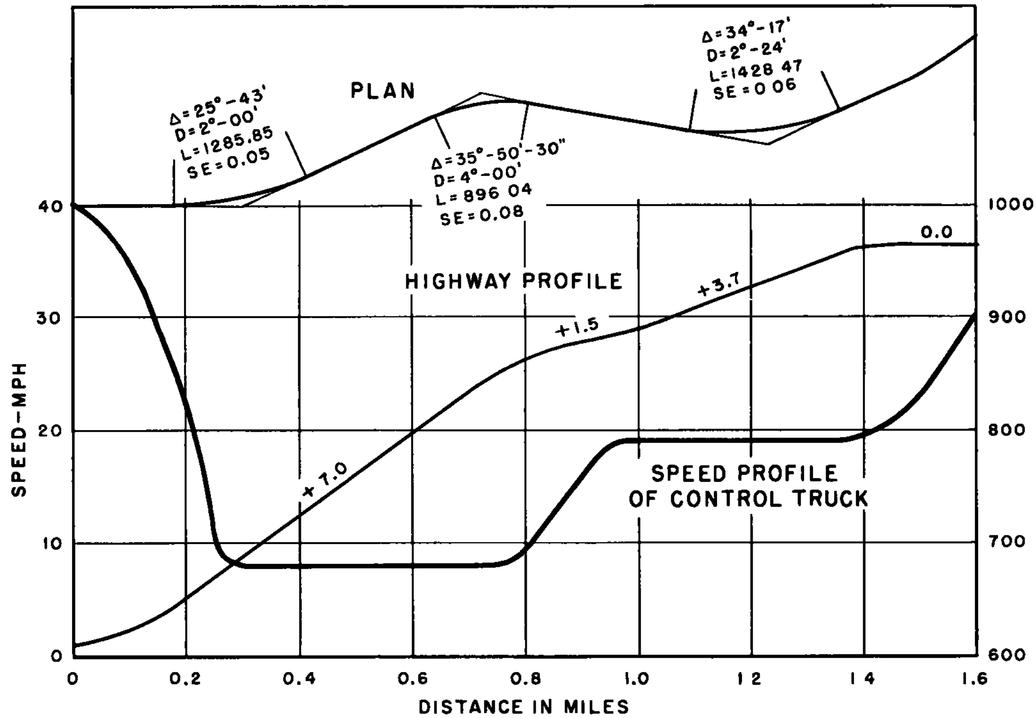

Figure 9. Speed profile of control truck.

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 by Figure 9, which also gives the speed profile recorded for the control truck.

TRUCK CLIMBING LANES

Truck climbing lanes on the uphill side of long steep grades provide a means for improving the capacity of 2-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 2-lane highway. This results in higher truck factors and lower capacities for uphill sections of a 2-lane highway than

TABLE 7

SPEED CHARACTERISTICS OF CONTROL TRUCK ON UPGRADES[a]

Gradient, Percent	Crawl Speed[b]		Distance Upgrade, ft		
	Velocity, mph	Distance Upgrade, ft	34 mph (T.F.=3.0)	27 mph (T.F.=6.5)	19 mph (T.F.=13.8)
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

[a]When entering grade from level section at 40 mph.

[b]Speed which truck can maintain indefinitely.

for the level sections.

Where truck climbing lanes are provided, the truck factor becomes zero and the capacity of the normal section of the 2-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 2-lane highway to a value higher than that for the same alignment 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 the 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 2-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 on the following factors:

1. Traffic volume.
2. Percentage of trucks.
3. Length and steepness of grade.
4. Availability of passing sight distance.

Table 7 offers some guidance for the application of climbing lanes. Column 4, for example, shows the lengths of grade for an average truck speed of 34 mph or a truck factor of 3.0. At this average speed, even though about one-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 2-lane highway, 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 Column 4, Table 7.

Columns 5 and 6, Table 7, show lengths of grade on which there is the same relative

TABLE 8

CAPACITIES OF 2-LANE HIGHWAYS ON GRADES

(Based on a 30th highest hour of 12 percent of the average annual volume)
5 Percent Trucks

Grade, Percent	Length of Grade, ft	Average Annual Traffic Volume, vehicles per day							
		Avg. Hwy. Speed 70 Oper. Speed 50-55		Avg. Hwy. Speed 60-70 Oper. Speed 50		Avg. Hwy. Speed 60-70 Oper. Speed 45-50		Avg. Hwy. Speed 50-70 Oper. Speed 40-45	
		Without Truck Lane	With Truck Lane	Without Truck Lane	With Truck Lane	Without Truck Lane	With Truck Lane	Without Truck Lane	With Truck Lane
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

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 ft long as on a 7 percent grade 800 ft long.

The capacities of 2-lane highways on grades with and without truck climbing lanes are shown by Table 8 for the conditions applicable to West Virginia. The various groups shown for the length of grade (Col. 2) are purely arbitary, 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 used is believed to be consistent with the accuracy justified by the analyzed data.

Table 8 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 ft have the same capacities. Prior to traveling 4,000 ft upgrade, most trucks will have reached their crawl speeds (Table 7).

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 (3, 4). With passing bays the capacity of a 2-lane road would be greater than without the passing bays and for certain conditions might equal the capacities shown in Table 8 for the 2-lane roads with a truck lane. The maximum capacities with continuous truck lanes are actually higher than most of the values in Table 8. For Table 8 it was assumed that the capacity on a grade with a truck lane could not exceed the capacity of a 2-lane level section. The capacity with a truck lane falls below the capacity of a level section only on the long grades greater than 5 percent where downhill speeds of trucks traveling in the lower gears affect capacities.

APPLICATION TO CAPACITY DETERMINATIONS

The tables and charts 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.

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 2-lane roads can be determined from Tables 1, 4, and 8, and Figure 6. Figure 7 was also needed for the Kentucky and Tennessee studies, because 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-ft passing sight distance. In addition, a county line was the end of one section and the beginning of another.

EXAMPLES OF 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 given in Table 9.

Section 1 is located on U.S. 60 about 20 mi west of Charleston in Putnam County. It is 6.4 mi long with a 26-ft pavement in rolling terrain. It has an excellent passing distance as compared with most West Virginia roads, 59 percent of its length having a sight distance in excess of 1,500 ft. The average highway speed is 65 mph 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 of 15 percent of the ADT having 7 percent trucks.

The capacity of this section is 5,800 vehicles daily at an operating speed of 45 to 50 mph, or is 7,150 vehicles daily at a tolerable operating speed of 40 to 45 mph. Because U.S. 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 to 50 mph 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 another one-way roadway and using the existing lanes for the other direction of travel.

Section 2, on U.S. 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-mi section is located in rolling terrain about 20 mi north of Charleston. Both the alignment and profile are poor, resulting in a low design speed and almost no places where the sight distance is adequate

TABLE 9

CAPACITY ANALYSIS OF TYPICAL SECTIONS IN WEST VIRGINIA

Item	Sect. 1	Sect. 2	Sect. 3	Sect. 4	Sect. 5
(a) Known Conditions					
Location	U.S. 60	U.S. 21	U.S. 60	U.S. 50	U.S. 50
Length, mi	6.37	6.21	7.00	4.13	11.46
Terrain	Rolling	Rolling	Rolling	Rolling	Rolling
Avg. highway speed, mph	65	40	53	57	53
1.500-ft S.D., %	59	2	10	9	15
Surface width, ft	26	18	22	22	20
30th-hr factor, %	12	12	12	13	14
Truck speed, mph	40	32	29	25	24
Commercial veh, %	7	5	5	5	5
1955 ADT	5,500	2,300	2,400	2,400	2,600
Long grades	None	None	0.40 mi, 8%	0.50 mi, 7.5% 0.35 mi, 4% 0.60 mi, 2.5%	0.5 mi, 6.5% 1.2 mi, 5.0% 2.5 mi, 2.5%
Sharp curves			1 at 30 mph 1 at 35 mph 1 at 45 mph 2 at 50 mph	None	1 at 40 mph 1 at 45 mph 3 at 50 mph
(b) Determined Values					
Tolerable operating speed, Table 1, mph	40-45	40-45	40-45	40-45	40-45
Tolerable design speed, Table 1, mph	50	50	50	50	50
Tolerable capacity, Table 4, ADT	7,300	None	3,070	3,800	3,405
Width factor	1.00	0.70	0.86	0.86	0.77
30th-hr factor	1.00	1.00	1.00	0.92	0.86
Truck equivalent	2.0	4.0	5.0	8.0	9.0
Truck factor*	0.98	0.91	0.88	0.78	0.75
Corrected tolerable capacity, ADT	7,150	None	2,300	2,350	1,700

$$* \quad \frac{105}{(100-\%\ \text{tr}) + (\%\ \text{tr} \times \text{tr equiv})}$$

for passing. Pavement width is 18 ft and the truck equivalent is 4. The traffic volume during 1955 was 2,300 vehicles per day with the design hour being 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. Inasmuch as its average highway speed is only 40 mph, it cannot carry traffic at the tolerable speed of 50 mph during low volumes nor at 40 to 45 mph during the 30th highest hourly volume of the year. It therefore has no capacity for these speeds.

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

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

The tolerable operating speed for this highway is 40 to 45 mph, therefore the 35- to 40-mph operating speed would be inadequate and undesirable. The capacity at a minimum desirable operating speed with the alignment 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. 60 in Greenbrier County about 100 mi east of Charleston. The terrain is rolling over this 7.0-mi section, the average highway speed is 53 mph, about 10 percent of the highway has a 1,500-ft sight distance, and the truck equivalent is 5.

Tolerable operating speed for this highway is 40 to 45 mph. 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 mph, 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 1 mi of truck lanes to be added, resulting in a decrease in the over-all truck equivalent from 5 to 3. Minor improvement in the alignment would also provide additional passing sight distance so that a 1,500-ft 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 to 45 mph. At the normal rate of traffic growth this volume would not be exceeded for a period of 6 to 7 years. Thereafter it would be necessary to undertake major changes in the alignment or to provide a 4-lane highway in order to maintain the desired operating speed.

Section 4 has 11-ft traffic lanes and is located on U.S. 50 in Wood County. It is 4.1 mi long through rolling terrain. The alignment is fairly good, as the average highway speed is 57 mph and 9 percent of the highway has a 1,500-ft 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 speed of 40 to 45 mph, the capacity is 2,350 vehicles per day. This is slightly lower than the present volume.

Building a truck lane 1 mi long on a critical grade would reduce the truck equivalent to 3 and would increase the 1,500-ft 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 to 45 mph.

Some additional 1,500-ft sight distance could be obtained by increasing the sight on the inside of several curves by simply removing such obstructions 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-ft sight distance can be obtained in thi 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. 50 in Hampshire County in the northeastern part of the state. The section is 11.5 mi long with uniform design characteristics in the rolling terrain. The average highway speed is 53 mph, the surface width is 20 ft, and 15 percent of the highway has a 1,500-ft sight distance. The truck equivalent is 9. The prese ADT is 2,600 per day with a design factor of 14 percent including 5 percent trucks. Under these conditions, the capacity at an operating speed of 40 to 45 mph 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 ft to 24 ft would increase the capacity to 2,200 vehicles per day. Removal of the substandard curves will increase the average highway speed to about 55 mph 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 2.5 mi 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 others, such as the provision of truck lanes and substantially improving the passing sight distances, will have a major effect.

APPLICATIONS TO CONDITIONS IN KENTUCKY AND TENNESSEE

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 size and weight, 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 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 Virgin

Determining Average Highway Speed

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, an additional time and distance is required to accelerate back to the normal tangent speed. It was necessary, therefore, 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." 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 ft at the safe speed for the curve. For example, the 9-deg curves good for a design speed of 45 mph had an average length of 667 ft. Decelerating and accelerating from the 65-mph tangent speed required a total of 485 ft. On an average, a vehicle would be affected for a total distance of 1,152 ft, but the time lost was the same as if the speed was 45 mph for 915 ft and the tangent speed of 65 mph on the rest of the section. Likewise, for the 40-deg curves the equivalent distance at 20 mph was 691 ft. The equivalent distances varied from curve to curve, but the average was 780 ft with values much greater or less than the average being comparatively rare. An equivalent length of 800 ft, or 0.15 mi, 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 deg were assumed to have a highway speed of 70 mph if there were no curves as sharp as 4 deg on the highway. If any curves on the highway were as sharp as 4 deg, the tangent sections and the curves of 4 deg or flatter were assumed to have a highway speed of 65 mph. 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.

The following example illustrates the method used in estimating the average highway speed of a 2-lane section of highway 10 mi long:

Curvature, deg	Safe Speed, mph	Number of Curves	Total Length, mi	Col. 2 x Col. 4
6	55	1	0.15	8.25
10	43	2	0.30	12.90
12	40	8	1.20	48.00
20	35	4	0.60	21.00
30	25	1	0.15	3.75
0	65	-	7.60	494.00
Total		16	10.00	587.90

$$\text{Average highway speed} = \frac{587.90}{10} = 59 \text{ mph.}$$

If weighted by travel time, the average highway speed would be 56 mph. Within the limits of reasonable accuracy, however, either method should be satisfactory. For the needs studies conducted in Ontario, Canada, weighting to obtain the average highway speed was done on the basis of time involved rather than length.

DETERMINING TRUCK EQUIVALENT

Attention was called earlier to the fact that driving a test truck to establish a speed profile would be unnecessary if gradient and length were known, because available test data are adequate to establish truck speeds on known grades (Fig. 8).

With grade data available in Kentucky and Tennessee, determining the truck equivalent in terms of passenger cars, for capacity computations, made use of Figures 7 and 6, in that order.

It was first assumed that the entering speed of trucks approaching a grade was 40 mph. 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. 50 in West Virginia shows a maximum of only 45 mph for short distances at only three locations in a 50-mi section.

It was also assumed for the purposes of this study, that average truck speed was 40 mph on level terrain, on all grades of less than 3 percent, and on grades of 3 percent less than 500 ft long.

For all other grades, the average truck speed was determined from 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 determine by weighting by distance the speeds on level terrain and the several grades.

Finally, the weighted average truck speed was entered on 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 "Manual of Engineering Procedure for Determining Needs of the Rural State Highway System," published by the state highway departments of Kentucky and Tennessee in 1954

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Appendix

TABLE A

TOLERABLE CAPACITIES OF EXISTING TWO-LANE ROADS, FLAT TERRAIN, OPERATING SPEED 45 TO 50 MPH, 5 PERCENT TRUCKS

Percent of Highway with Passing Sight of		Average Annual Volume, Vehicles per Day															
		12-ft Lanes				11-ft Lanes				10-ft Lanes				9-ft Lanes			
		passenger car speed (average highway speed) at low volume															
1,500 ft	1,000 ft	70 mph	60 mph	55 mph	50 mph	70 mph	60 mph	55 mph	50 mph	70 mph	60 mph	55 mph	50 mph	70 mph	60 mph	55 mph	50 mph
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 ROADS, FLAT TERRAIN OPERATING SPEED 40 TO 45 MPH, 5 PERCENT TRUCKS

Percent of Highway with Passing Sight of		Average Annual Volume, Vehicles per Day															
		12-ft Lanes				11-ft Lanes				10-ft Lanes				9-ft Lanes			
		passenger car speed (average highway speed) at low volume															
1,500 ft	1,000 ft	60 mph	55 mph	50 mph	45 mph	60 mph	55 mph	50 mph	45 mph	60 mph	55 mph	50 mph	45 mph	60 mph	55 mph	50 mph	45 mph
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 ROADS, ROLLING TERRAIN OPERATING SPEED 45 TO 50 MPH, 5 PERCENT TRUCKS

Percent of Highway with Passing Sight of		Average Annual Volume, Vehicles per Day															
		12-ft Lanes				11-ft Lanes				10-ft Lanes				9-ft Lanes			
		passenger car speed (average highway speed) at low volume															
1,500 ft	1,000 ft	70 mph	60 mph	55 mph	50 mph	70 mph	60 mph	55 mph	50 mph	70 mph	60 mph	55 mph	50 mph	70 mph	60 mph	55 mph	50 mph
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 ROADS, ROLLING TERRAIN OPERATING SPEED 40 TO 45 MPH, 5 PERCENT TRUCKS

Percent of Highway with Passing Sight of		Average Annual Volume, Vehicles per Day															
		12-ft Lanes				11-ft Lanes				10-ft Lanes				9-ft Lanes			
		passenger car speed (average highway speed) at low volumes															
1,500 ft	1,000 ft	60 mph	55 mph	50 mph	45 mph	60 mph	55 mph	50 mph	45 mph	60 mph	55 mph	50 mph	45 mph	60 mph	55 mph	50 mph	45 mph
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 ROADS, ROLLING TERRAIN OPERATING SPEED 35 TO 40 MPH, 5 PERCENT TRUCKS

Percent of Highway with Passing Sight of		Average Annual Volume, Vehicles per Day															
		12-ft Lanes				11-ft Lanes				10-ft Lanes				9-ft Lanes			
		passenger car speed (average highway speed) at low volumes															
1,500 ft	1,000 ft	55 mph	50 mph	45 mph	40 mph	55 mph	50 mph	45 mph	40 mph	55 mph	50 mph	45 mph	40 mph	55 mph	50 mph	45 mph	40 mph
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 ROADS, MOUNTAINOUS TERRAIN OPERATING SPEED 45 TO 50 MPH, 5 PERCENT TRUCKS

Percent of Highway with Passing Sight of		Average Annual Volume, Vehicles per Day															
		12-ft Lanes				11-ft Lanes				10-ft Lanes				9-ft Lanes			
		passenger car speed (average highway speed) at low volumes															
1,500 ft	1,000 ft	70 mph	60 mph	55 mph	50 mph	70 mph	60 mph	55 mph	50 mph	70 mph	60 mph	55 mph	50 mph	70 mph	60 mph	55 mph	50 mph
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 ROADS, MOUNTAINOUS TERRAIN OPERATING SPEED 40 TO 45 MPH, 5 PERCENT TRUCKS

Percent of Highway with Passing Sight of		Average Annual Volume, Vehicles per Day															
		12-ft Lanes				11-ft Lanes				10-ft Lanes				9-ft Lanes			
		passenger car speed (average highway speed) at low volumes															
1,500 ft	1,000 ft	60 mph	55 mph	50 mph	45 mph	60 mph	55 mph	50 mph	45 mph	60 mph	55 mph	50 mph	45 mph	60 mph	55 mph	50 mph	45 mph
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	2,600	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 ROADS, MOUNTAINOUS TERRAIN OPERATING SPEED 35 TO 40 MPH, 5 PERCENT TRUCKS

Percent of Highway with Passing Sight of		Average Annual Volume, Vehicles per Day															
		12-ft Lanes				11-ft Lanes				10-ft Lanes				9 ft Lanes			
		passenger car speed (average highway speed) at low volumes															
1,500 ft	1,000 ft	55 mph	50 mph	45 mph	40 mph	55 mph	50 mph	45 mph	40 mph	55 mph	50 mph	45 mph	40 mph	55 mph	50 mph	45 mph	40 mph
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

Operation of Weaving Areas

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● A KNOWLEDGE of the capacities of weaving sections has become an extremely important requirement in modern highway design and traffic operation, especially for freeways, channelized intersections, and other designs where the crossing of two or more traffic streams is not controlled by traffic signals. Failure to recognize the existence of a weaving section or to predetermine its effect on traffic movement has resulted in unsatisfactory operating conditions at numerous locations on freeways opened to traffic during the past several years. At most of the locations, reconstruction has already corrected the unsatisfactory condition, or plans are being made to correct the condition. In each case, the cost of correcting the condition has been high compared to what the added cost would have been to eliminate the unsatisfactory condition during the original design and construction.

Accurate traffic counts by vehicle types for each of the movements through several of these problem locations have been recorded for each 10- or 15-minute time interval, together with the operating conditions, congestion, and vehicle speeds during the peak and nearpeak periods of flow. This report is based on the results of an analysis of these data.

A weaving section is defined in the "Highway Capacity Manual" (1) as: "A length of one-way roadway serving as an elongated intersection of two one-way roads crossing each other at an acute angle in such a manner that the interference between cross traffic is minimized through substitution of weaving for direct crossing of vehicle pathways." Highway sections that specifically meet this definition are generally easily recognized.

In its broader sense, however, the definition also includes a large variety of highway sections not so easily recognized, such as the roadway between two inner loops of a cloverleaf, where the vehicles entering a freeway from the one inside loop must cross the path of the vehicles leaving the freeway on the other inside loop. Likewise, any section of a freeway between an "on ramp" which precedes an "off ramp" is in effect a weaving section, although it is generally not recognized as such unless the two ramps are close together. Such a condition does, however, affect both the possible and practical capacities of the section whenever the percentage of vehicles that must merge and shift lanes per unit length of highway exceeds the percentage of vehicles that normally shift lanes on sections far removed from any entrances or exits.

Studies conducted during high volume conditions on freeways show that about 10 percent of the vehicles shift lanes within each 1,000 ft for reasons other than those connected with entrance and exit requirements. To this extent, the effect of vehicles shifting lanes is therefore included in normal capacity determinations of multilane facilities. Whenever the required amount of lane shifting between an entrance and an exit exceeds this normal rate, the section of highway may be considered as a weaving section in a capacity analysis. For example, if 10 percent of the traffic on a section of highway entered from a ramp at one end and another 10 percent left the section at the other end, the section should be checked as a weaving area in any capacity determinations unless the two ramps are more than 2,000 ft apart. For most such conditions the analysis will show that the problem is simply one of merging two traffic streams; but under certain conditions, depending on the distribution of traffic between lanes, which varies with the total traffic, the percentage of trucks, and the design of the ramps, the problem is the same as for a weaving area.

Figure 1, showing the results of capacity studies conducted to determine operating characteristics of weaving sections, is similar to Figure 43 of the "Highway Capacity Manual" (1), the difference being that the curves entitled "Maximum possible capacity" and "30-mph operating speed" are somewhat higher than the corresponding curves in the original publication.

The results of recent studies conducted at weaving sections, including several of the sections on which the original curves were based, show that the maximum possible capacities of weaving sections are now somewhat higher than they were about 10 years ago when the original figure was prepared. The reason for this increase cannot be determined, but it may be due in a large measure to improved driver performance as a result of increased experience or practice in driving through weaving areas.

Other considerations are, however, more important than the difference between the values in the original publication and those of Figure 1 that are based on a larger volume of data and more current information. These more important considerations include:

1. The correction of improper methods which have been used in some instances when applying the data for weaving sections.
2. Refinements that have been developed which permit a better agreement between values estimated by the use of Figure 1 and the actual field conditions.
3. New techniques which have been developed for the analysis of weaving sections with more than two entrances or exits.

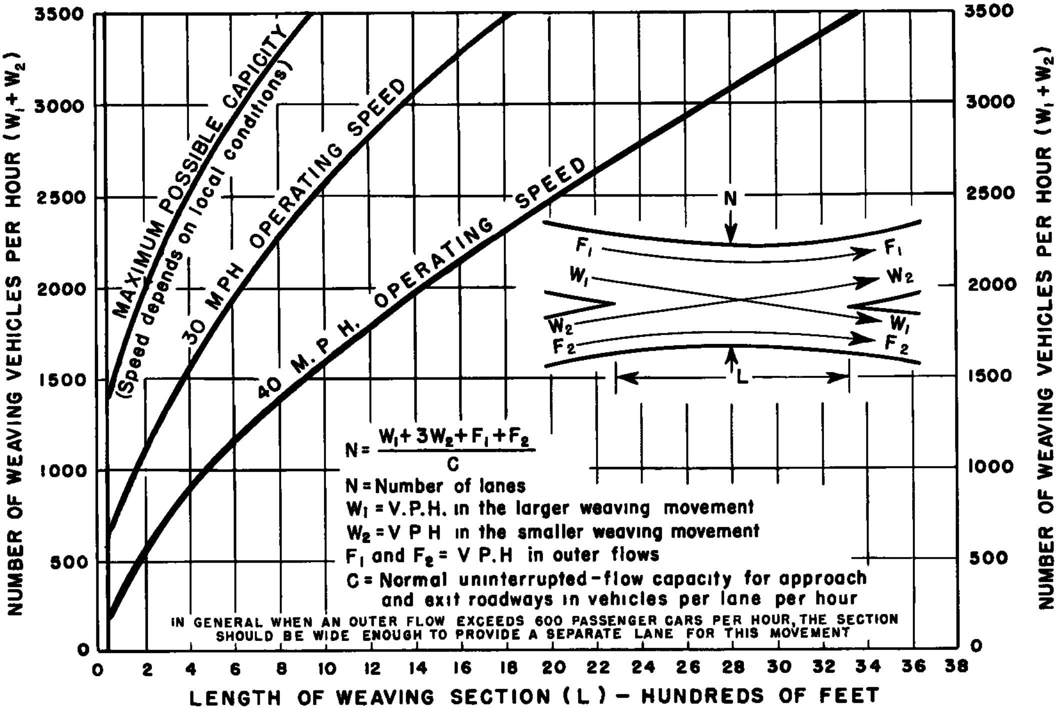

Figure 1. Operating characteristics of weaving sections.

The following are a number of items to clarify the use of Figure 1:

1. Weaving sections that satisfy both the length and width requirements as indicated by the chart will accommodate at least the corresponding traffic volumes. Because it was believed that the information would be used principally for design purposes, it was necessary to be sure that the resulting actual capacity would be at least as high as calculated. The instances during the past ten years in which accurate studies have revealed errors of a significant magnitude have been few, and in all such cases the actual volumes have been greater than calculated, an expected result.

2. The curves for the 30- and 40-mph operating speeds are applicable to the speeds of the vehicles that must perform the weaving maneuvers. The speeds of the other vehicles would depend on the value of C used. When weaving vehicles negotiate the section at a higher average speed during the corresponding traffic volumes, it is generally at the expense of the non-weaving traffic.

3. The 40-mph curve was intended principally for use in rural areas where practical lane capacities of 1,000 vehicles per hour (C) on the normal sections will assure

an operating speed of 45 to 50 mph; the 30-mph curve was intended for urban areas where practical lane capacities of 1,500 vehicles per hour (C) on the normal sections will assure operating speeds of 35 to 40 mph. The resulting slowdown at the weaving section for the vehicles involved in the weaving maneuvers was considered reasonable for these conditions.

Where higher or lower speeds than these occur on the approach roadways for the corresponding traffic volumes, the speeds of the weaving vehicles may also be expected to be correspondingly higher or lower. For example, when the operating speeds are 40 to 45 mph on the normal sections of highway when the volume is 1,500 vehicles per lane, the speeds of the weaving vehicles as selected from the 30-mph curve can be expected to be 35 rather than 30 mph and if 1,500 is used as the value of C, vehicles not involved in the weaving can be expected to negotiate the section at an average speed of 40 to 45 mph. This would, however, be an unusual condition, because at most locations speeds on the normal sections are 35 to 40 mph at volumes of 1,500 vehicles per lane.

4. It should be noted that no speed value has been placed on the curve for maximum possible capacity. This is because some sections attain their maximum capacity with the faster weaving vehicles averaging close to 30 mph, whereas at others the maximum capacity is attained with stop-and-go operation by nearly all the weaving vehicles. The same location may in fact accommodate the same maximum volume under several different operating conditions, but in each case the average speed of the weaving vehicles is below 30 mph. It is not correct, therefore, to assume that the higher volume curve is for 20 mph and that values for intermediate speeds can be obtained by interpolation.

5. In applying the equation for the number of lanes, the value of C must be adjusted for lane width and other factors (such as trucks) in the same manner as for other free-flowing facilities in determining both practical and possible capacities.

6. The note at the bottom of Figure 1 which reads: "In general when an outer flow exceeds 600 passenger cars per hour, the section should be wide enough to provide a separate lane for this movement," is not intended to imply that one lane should be added for each such movement in addition to the number, N, calculated by the formula. It was the intent that when separate lanes are provided for outer flows in excess of 600 vph, the appropriate symbol (F_1, or F_2, or both) would be omitted from the equation for determining the number of lanes. The lanes provided separately for outer flows would then be added to the number required for the weaving vehicles in order to obtain the total number of lanes in the section. It is only when N is less than 3 for a total flow with one outer movement exceeding 600 vph, or N is less than 4 for a total flow with both outer movements exceeding 600 vph, that one additional lane is recommended for each outer flow above 600 vph.

7. The relationship between the length and width of the weaving section as shown by Figure 1 will result in the minimum number of traffic lane miles within the weaving section to effect the crossing of the two traffic streams with an assurance of attaining the desired operating conditions. In many instances it is possible or more feasible to provide weaving sections longer than the needed lengths as shown by Figure 1. An adjustment may then be made in the width of the weaving section, because the necessary weaving maneuvers may be made over a longer distance. The adjustment is made in the term $3W_2$ and is based on the criterion that the added gaps needed for the weaving maneuvers vary inversely as the ratio between the actual length and the minimum required length as shown by Figure 1. For example, if Figure 1 shows that a weaving section 500 ft long is required, but a 1,000-ft section is provided, the term $3W_2$ becomes

$$\left[\frac{2\ (\text{Length as shown by Fig. 1})}{\text{Actual length}} + 1\right] W_2$$

or $2W_2$. Likewise, if the section were actually three times as long as indicated by Figure 1, the term $3W_2$ would become $1.7W_2$.

Comprehensive studies are now under way to determine the effect of ramp design and the spacing between interchange ramps on freeway traffic operations. Until the results of these studies become available, it will not be possible to establish definite criteria governing the effect that weaving maneuvers between "on" and "off" ramps widely spaced have on the capacities of the intervening roadways. Lane usage or the distribution of traffic by vehicle type in the various lanes, the normal shifting between lanes because of speed differentials, and the lengths of the acceleration and deceleration lanes are but a few of the factors involved.

Oftentimes, on expressways, a weaving section may have more than two entrances or exits adjacent to one another or so closely spaced as to form a compound weaving section, or two weaving sections that overlap. When there are two entrances and three exits, or three entrances and two exits, there may be six different movements rather than four as in a weaving section with two entrances and two exits. Although traffic operation studies have been conducted at few such locations, the results indicate that Figure 1 can be applied by employing a somewhat modified treatment.

The required length of a section with three entrances or exits adjacent to one another may be determined by using the total number of weaving maneuvers that must be performed. In calculating the total number of lanes required the number of vehicles involved in the minor movements, wherever two movements cross, are tripled and added to the major crossing movements. For example, if the two entrances from left to right are A and B and the exits from left to right are X, Y, and Z, the two outer movements not involved in weaving would be AX and BZ. The maneuvers involved in weaving would be AY, AZ, BX, and BY. AY would cross BX, and AZ would cross both BX and BY. The number of weaving maneuvers in calculating the required length of the weaving section would become AY + BX + 2AZ + BX + BY, or AY + 2BX + 2AZ + BY. In calculating the number of lanes required, N, when AY > AZ > BX > BY, the numerator of the equation becomes AX + BZ + AY + 3BX + AZ + 3BX + 3BY + AZ, or AX + BZ + AY + 2AZ + 6BX + 3BY.

When the three entrances or the three exits are not adjacent to one another, the same principles may be applied by separating the overlapping weaving sections and making separate calculations for each. There is, in effect, a long weaving section which overlaps a shorter weaving section. The capacity analysis may be performed by separting the overlapping sections into two separate weaving areas and applying the principles previously discussed.

The weaving maneuvers must, however, be separated into two groups: (a) those that must be performed in the shorter section and (b) those that may be performed at any point over the entire section.

To check a fixed design, as many as possible of the weaving maneuvers in the second group are assigned to the area of the longer section which does not overlap the shorter section. Then those in the first group, and the balance of the second group, must be accommodated in the shorter section. Such a procedure could be followed to check the capacity of the example previously used to illustrate a weaving section with three exits, if exit X or exit Z occurred in advance of the other two exits and the distances between entrances and exits were fixed. When these distances are not fixed, several repeat calculations must be made to obtain a reasonable design, because there would be a large variety of lengths that could be employed in combination with different widths.

REFERENCE

1. "Highway Capacity Manual," U.S. Govt. Printing Office, Washington, D.C. (1950).

Pressurized Intersections

A STUDY OF INTERSECTION APPROACH CAPACITIES ON UNDIVIDED STREETS IN AN INTERMEDIATE-TYPE AREA

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Bruce Campbell and Associates, Boston, Massachusetts

This report on capacities of signalized intersections is the first part of a series of comparison studies between results presented in the 1950 issue of the Highway Capacity Manual and results as found for specific "pressurized" intersections in the Boston Metropolitan Area.

Data from 21 pressurized intersections (that is, intersections where the approach under study had a continual backlog of waiting vehicles) indicated capacities in excess of those reported in the "Highway Capacity Manual." Specifically, volumes on approaches with a total street width of 31 to 46 ft averaged 14 percent higher than "Manual" averages. On streets 47 to 64 ft in total width, volumes were 65 percent greater; on streets 65 ft and wider, the average volumes were 106 percent higher than corresponding volumes in the manual.

It is not intended that the results of the study be used as general criteria for higher basic design volumes. Rather it indicates a much higher capacity potential, especially in the wider street width, under pressurized conditions which motorists are currently tolerating in larger metropolitan areas.

The reductive effect of commercial vehicles and turning movements under pressurized conditions was compared with those in the manual. The factor found locally to have the greatest influence on the capacities was the turning movement. Left turns appeared to have a greater initial effect under pressurized conditions than the manual indicates. However, this effect appears to lessen on approaches carrying more than 20 percent of left-turning traffic. Also, the local data indicate that when left-turning volumes are equal to or greater than 50 percent of the total approach, they operate as through traffic with little reductive effect. This is believed to be peculiar to intersections where the major movement is the left turn and the peak condition involves primarily the same daily commuter traffic.

The effect of commercial vehicles was found consistent with that proposed by the manual (in the range of 0 to 20 percent). The data indicate a greater effect than that outlined in the manual due to the presence of commercial vehicles when they make up more than 20 percent of the total approach volume.

Finally, the study indicates that the reductive effect of the opposing left turn is sufficient to be evaluated alongside the usual lefts, rights, and commercial percentages.

●THE APPROACH CAPACITY of a signalized intersection is undoubtedly the greatest retardant to the basic capacity of a highway or street. When the intersection approach has become continuously backlogged with waiting vehicles, the intersection becomes "pressurized" with impatient vehicles.

The Highway Capacity Manual (1) has attempted to determine the capacity of various intersectional approaches by summarizing and plotting a wide variety of field measured data. The analytical study of the manual consists of classifying intersections in accordance with their geographical location; downtown areas, intermediate areas, and rural or outlying areas. Each location is considered with both parking prohibited and parking permitted. The classes are further sub-divided into two-way, one-way and divided type streets.

Having standardized the intersections, the actual traffic volumes are then standardized in order to compare relative conditions. This is done by adjusting the various

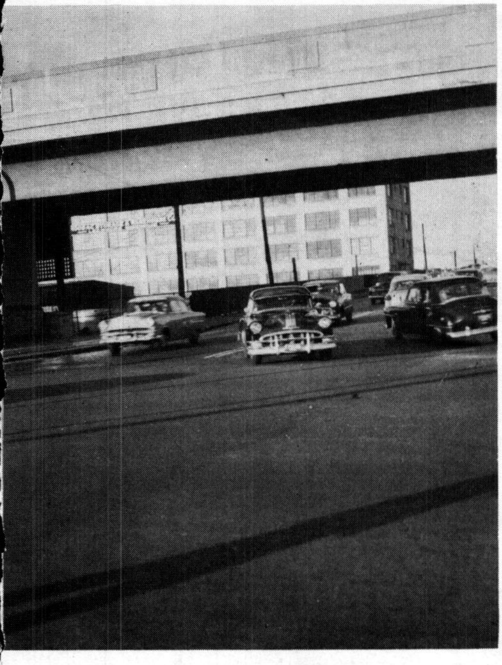


Figure 1. Maximum possible capacity.

influencing factors to a common unit. The basic reference or "average condition" is 10 percent commercial, 10 percent right turns and 10 percent left turns. Characteristics differing from this are adjusted to the common unit of measure by a combined adjustment factor, obtained as follows: Subtract or add 1 percent for each 1 percent commercial traffic differing, subtract or add $\frac{1}{2}$ percent for each 1 percent of right turns differing, subtract or add 1 percent for each 1 percent of left turn differing; and subtract 10 percent for bus stops on the near side, subtract 15 percent for bus stops on far side, or add 5 percent if no bus stops. For intersections with unusual conditions, the adjustments are subdivided in more detail as outlined in the manual.

With the intersection capacity depending upon the approach capacity, the approach capacity, in turn, depends primarily upon two fundamental features, time and space. The time is measured in seconds of green signal, or by subdividing the cycle and phase lengths per hour of total elapsed time. The space is measured in units of 10-ft lanes. The capacity is therefore expressed in units of vehicles per 10 ft width per hour of green signalization.

The intended unit of field measure was the "possible capacity," the maximum number of vehicles that actually can be accommodated under prevailing conditions with a continual backlog of waiting vehicles (Fig. 1). O.K. Normann summarizing the data in the manual concluded that not all data met this requirement. Hence, it was determined that the average condition (as plotted for Curve 1, Fig. 8) was midway between the possible capacity and a "practical capacity" — practical capacity defined as the

Figure 2. Practical capacity.

Figure 3. Congested intersection with continuous backlog of vehicles.

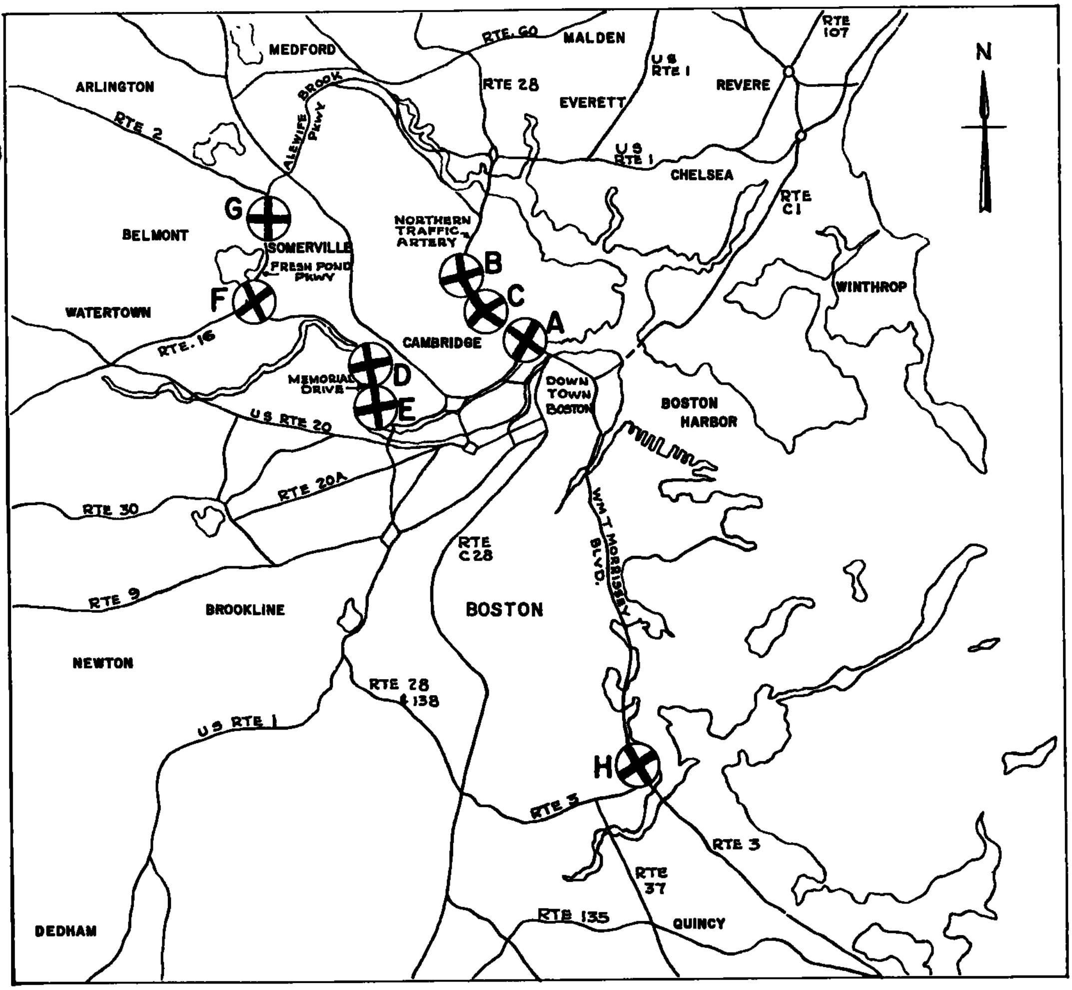
maximum volume that can enter from an approach during one hour, with most of the drivers being able to clear the intersection without waiting for more than one complete signal cycle (Fig. 2).

Figure 4. Congested intersection with continuous backlog of vehicles.

PRESSURIZED INTERSECTIONS

BOSTON METROPOLITAN AREA

LOCATION MAP



KEY

LETTER	INTERSECTION
A	NORTHERN TRAFFIC ARTERY & PRISON POINT BRIDGE
B	" " " & WASHINGTON ST
C	" " " & SOMERVILLE AVE
D	MEMORIAL DRIVE & RIVER ST
E	" " & WESTERN AVE.
F	FRESH POND PARKWAY & HURON AVE.
G	ALEWIFE BROOK PARKWAY & RINDGE AVE
H	WM. T. MORRISSEY BLVD & REDFIELD ST.

Figure 5.

This report has derived the average condition by dividing actual possible capacity field counts by a capacity load factor of 1.1 or dividing actual practical capacity field counts by a factor of 0.9. The actual field counts, after being divided by the combined adjustment factor and by the capacity load factor, are plotted against the total street width for the intermediate type areas.

Basis capacity was adopted as 1,250 vehicles per 10 ft width per hour of green, stemming from the basic capacity of 1,500 vehicles per 12 ft width per hr for 1 lane of continuous moving traffic at 12 mph.

SELECTION OF INTERSECTION APPROACHES

Twenty-one intersection approaches of the undivided, 2-way intermediate class were selected for study. Each was known to carry large volumes of peak hour traffic (Figs. 3 and 4). The approaches were part of eight intersections (Fig. 5) widely dispersed over the Metropolitan Area. Each was definitely an intermediate type intersection, located between the downtown area and the outlying residential areas. During peak hours, they served primarily commuting traffic, and during off-peak hours served considerable cross-town traffic. They varied in total width from 30 to 94 ft, and contained a variety of traffic type and movement. There were no streetcars, nor was parking permitted on any approach.

COLLECTION OF DATA

The first method of data collection was by the use of standard one-half hour traffic counts, as available on office file (Fig. 6). The traffic count furnished actual volumes of passenger vehicles, commercial vehicles and turning movements per period of one-half hour elapsed time. Additional office files furnished dial timing of signals, intersection dimensions and other relevant information. All file data used was less then one year old.

The second method of data collection served as a check on the first. It consisted of using special phase counting forms, similar to those used by the U.S. Bureau of Public Roads for collection of data (Fig. 7). Five intersection approaches were double checked in this manner. This method of traffic count furnished traffic volumes and turning movements for each green phase of the signal, and specifically indicated those phases that had a continuous backlog of waiting vehicles. By grouping a series of these fully loaded phases, and omitting any phase not fully loaded, volumes of average maximum possible capacity were found per unit of green time.

SAMPLE CALCULATIONS

The eastbound approach on Washington Street at the Northern Artery, Somerville, Mass. (Table 1, line 8) will be used in demonstrating the method used in the calculations.

The physical characteristics were (a) intermediate type intersection, (b) street width of 80 ft at approach, and (c) parking prohibited on both approach and exit.

The traffic characteristics were (a) count taken on Friday, 3:00 - 4:00 p.m., (b) continuous backlog of waiting vehicles, (c) traffic count of 882 vehicles per hour, (d) 35 percent commercial traffic, (e) 1.6 percent right turns, (f) 52 percent left turns, and (g) bus stops at curb on near side.

The signalization characteristics were (a) 140 sec, three-phase signal, (b) the apprach is on a separate phase, and (c) 33 sec green per cycle.

Adjustments were as follows:

Influence		Adjustment (percent)	Factor
35 percent commercial	(10-35)	-25	0.75
1.6 percent right turns	(10-1.6)½	+ 4.2	1.042
52 percent left turns	(10-0)	+10	1.10
Bus stops		-10	0.90

Because of intersection design and volume left turns were treated as through traffic. The combined adjustment factor (0.75 x 1.042 x 1.10 x 0.90) = 0.77. The capacity load factor, measured at maximum possible capacity, was 1.1.

COMMONWEALTH OF MASSACHUSETTS METROPOLITAN DISTRICT COMMISSION

PARKS DIVISION

TRAFFIC MOVEMENT SUMMARY TABLE

LOCATION ______________________ CITY OR TOWN ______________________

DATE ____________ DAY OF WEEK ____________ WEATHER ____________ RECORDER ______ STA.NO. __

OVER FOR SKETCH

TIME STARTS ______M	BOUND ON ST			BOUND ON ST.			BOUND ON ST			BOUND ON ST			TOTAL HALF HOURLY TALLY	
	L	S	R	L	S	R	L	S	R ·	L	S	R		
700-730														
730-800														
800-830														
830-900														
900-930														
930-1000														
1000-1030														
1030-1100														
1100-1130														
1130-1200														
1200-1230														
1230-100														
100-130														
130-200														
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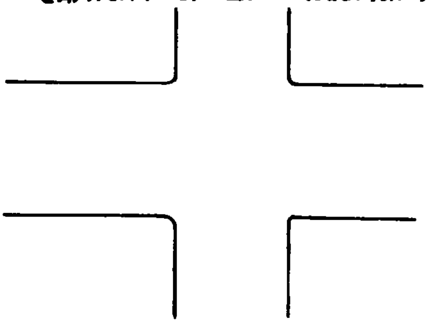
300-330														
330-400														
400-430														
430-500														
500-530														
530-600														
600-630														
630-700														
700-730														
730-800														
800-830														
830-900														
900-930														
930-1000														
1000-1030														
1030-1100														
TOTAL													GRAND TOTAL	
TOTAL OF L.S & R														

Figure 6.

SHEET No ____

CAPACITY OF INTERSECTION APPROACH ST. PER GREEN TIME



IF PARKING PERMITTED, CROSS-HATCH ( )

IF BUS STOPS PERMITTED, PLACE "B" AT STOP LOCATION

PLACE ARROW FOR EACH LANE MOVEMENT ( )

LIST SEPERATE TURNING ARROW DATA ON SEPERATE LINE

SKETCH UNUSUAL CIRCUMSTANCES

________ BOUND ON ____________ ST.
AT ____________ ST.

STARTING TIME ____________ M
FINISHING TIME ____________ M

SEC GREEN	CARS			COMMERCIAL			✓ IF LOADED	SEC GREEN	CARS			COMMERCIAL			✓ IF LOADED
	L	S	R	L	S	R			L	S	R	L	S	R	

Figure 7.

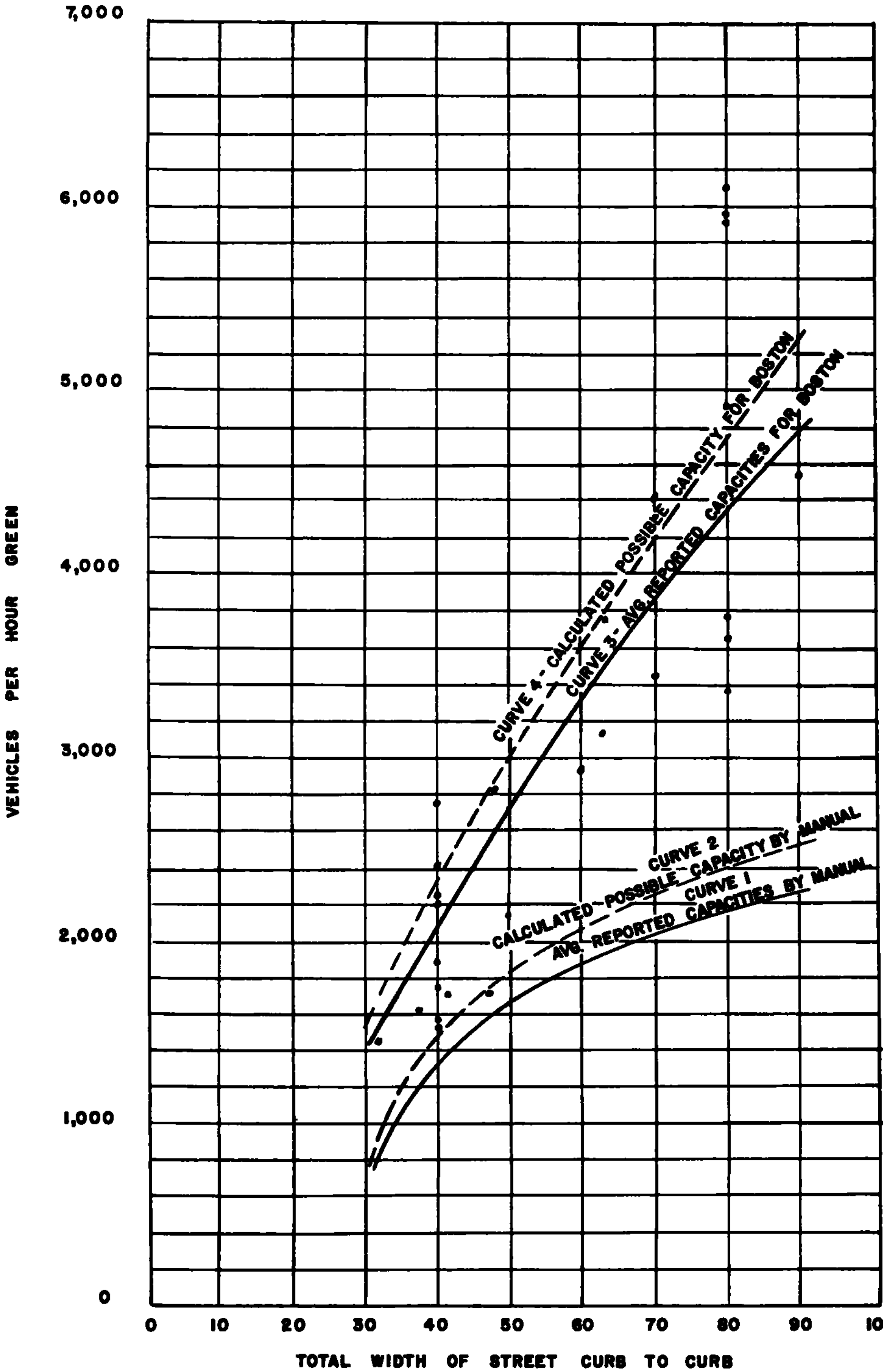


Figure 8. Average reported intersection capacities for two-way intermediate type streets with parking prohibited.

The manual reports the average capacity for an 80-ft street as 2,180 veh per hr of green when under an average condition of 10 percent commercial, 10 percent left turns and 10 percent right turns (Fig. 8). Therefore the possible capacity of the above approach would be: 2,180 x 33/140 x 0.77 x 1.1, equaling approximately 435 vehicles per hour of time. The actual field counts indicated that 882 veh per hr were passing through the intersection under the above conditions.

Knowing the existing capacity of the approach, the calculations may be reversed in order to solve for an adjusted capacity, comparable to the average condition presented by the manual, or 882 x 140/33 x 1/0.77 x 1/1.1 equals 4,420 vehicles per hour green

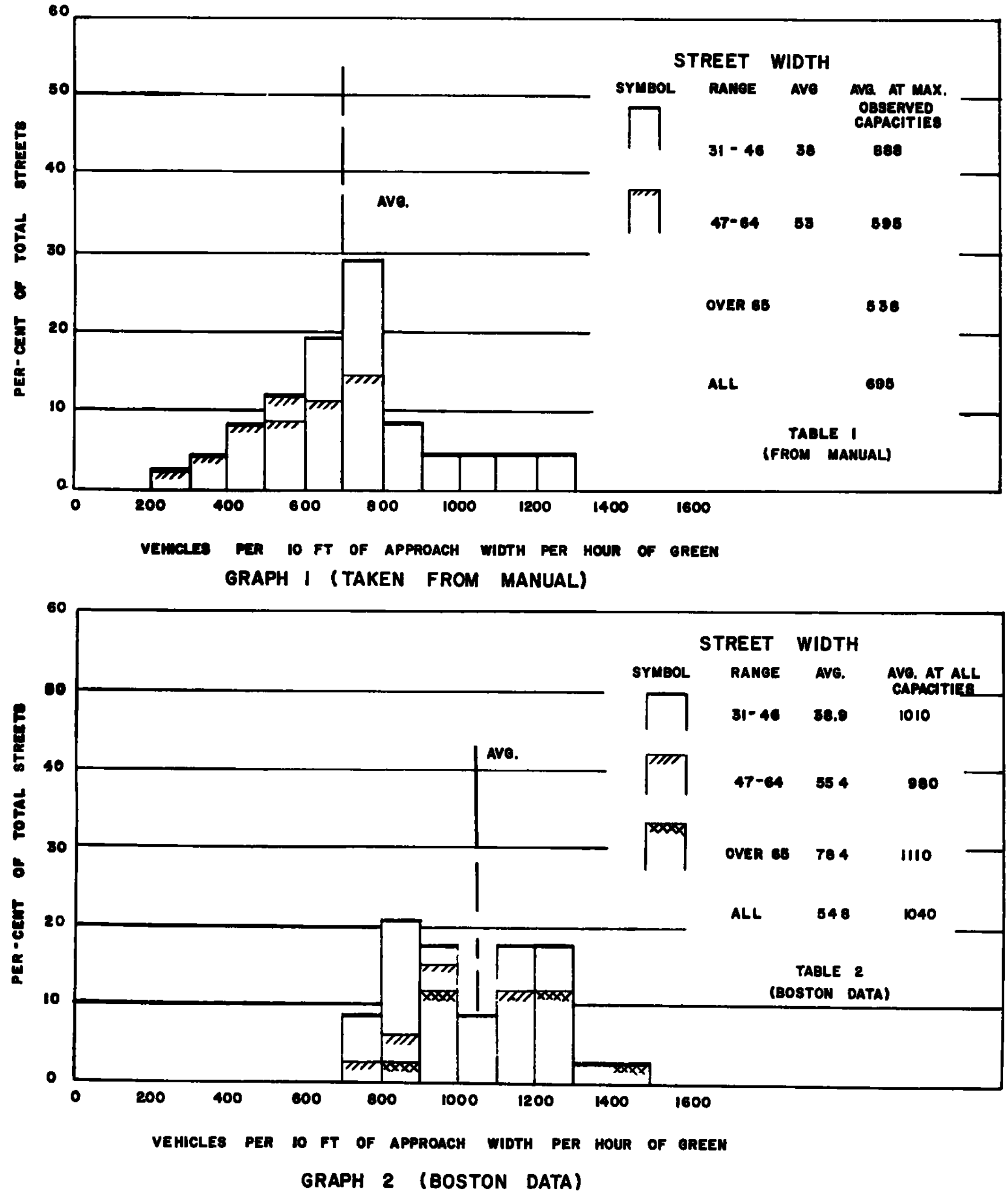


Figure 9. Frequency distribution of intersection capacities for intermediate type areas with parking prohibited.

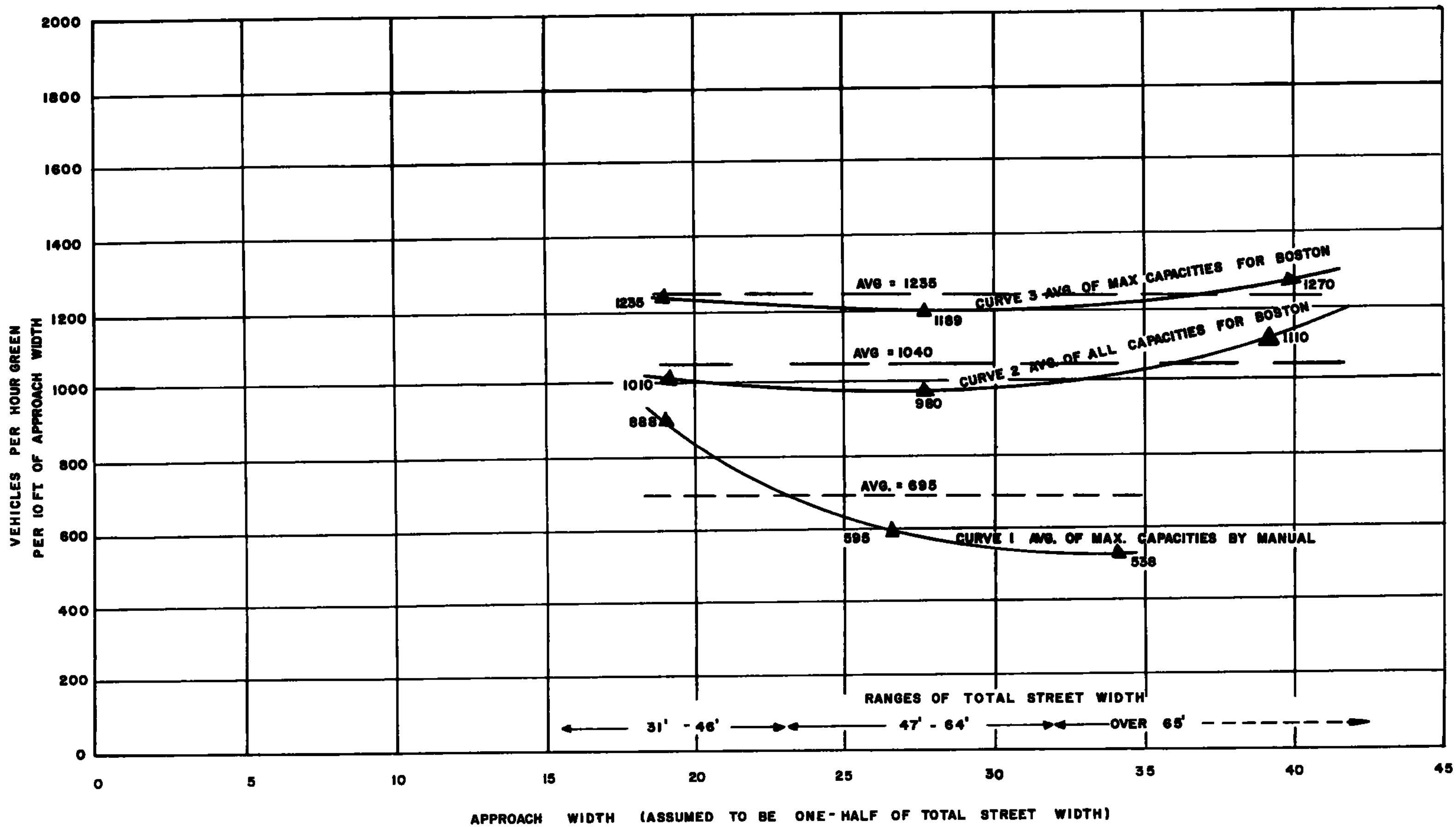


Figure 10. Average maximum reported approach capacities per range of street width.

at average conditions, thus, 4,420 veh per hr green per 80-ft total width, or 550 veh per hr green per 10-ft total street width, or 1,105 veh per hr green per 10-ft approach width.

PLOTTED DATA

Figure 8 is a direct comparison of average reported capacities. Curve 1 is the average reported capacities of many U.S. cities and has been taken from the Capacity Manual for purposes of comparison. Curve 2 is the maximum possible capacity of the reported cities, and is obtained by multiplying the average condition by the load factor of 1.1.

Curve 3 is a mean plot of the raw data for Boston as presented in this study. Curve 4 is the maximum possible capacity obtained by multiplying the average capacity of Curve 3 by 1.1.

Curves 3 and 4 indicate two significant findings: (1) the average reported capacities are considerably higher than expected, and (2) the capacity of an approach increases almost in direct porportion to its increase in width. A street 80 ft in total width carries over twice the approach volume of a street 40 ft in total width. This finding is not applicable for normal highway capacities, but instead probably results from the intensive pressure of backlogged traffic in a congested urban area.

Figure 9 indicates the frequency distribution of the intersection capacities found. Graphs 1 and 2 present a comparison between study data and data presented by the Capacity Manual. The total street widths have been sub-divided into common width ranges. The manual presents an average of maximum observed capacities, whereas the study presents the over-all average of all capacities as found. No precise determination has been made as to what determines the dividing line between maximum and average figures. In later comparisons, this study uses the upper one-third as maximums for comparable purposes.

The differences between the capacities shown in Figure 9 and the basic intersection capacity of 1,250 vehicles per hour of green per 10 ft of width, can be attributed primarily to the combind effect of the adjustments previously mentioned.

Graph 2 (Fig. 9) indicates that 100 percent of the approaches studied had an approach capacity of at least 800 vehicles per 10 ft of approach per hour of green, as compared to 27 percent of the approaches summarized by the Capacity Manual. This study has found an over-all average of 1,040 as compared to the manual's 695 vehicles per 10 ft of approach width per hour of green.

Figure 10 indicates the trend of maximum approach capacities in relation to the range of total street width. The capacities for each range of street width have been plotted against the approach width. Curve 1 indicates a plot of the average maximum capacities as summarized by the Capacity Manual, whereas Curve 3 is the average maximum capacities found in this study (upper one-third of each range). Curve 2 is a plot of the over-all average capacities as found for the width ranges.

The over-all average of this study is even higher than the maximum reported capacities of the manual.

Figure 11 indicates the trend of average reported capacities per 10 ft of approach width in terms of actual approach width, rather than ranges of width as previously shown. This data is obtained by dividing the plotted volumes of Figure 8 by the approach width. Approach widths of 5-ft increments have been used in establishing the plotted points. This figure not only indicates higher reported capacities, but indicates the range of approach width most efficiently used, 25 to 35 ft.

REDUCTIVE EFFECT OF TRAFFIC MOVEMENTS

Because of the very high volumes of traffic on the intersection approaches, a brief study was made in an attempt to evaluate and correlate the average reductive effect that each individual traffic movement has on the capacity of an intersection approach.

Nine of the intersection approaches previously studied and six additional ones were selected for further study. All approaches were 40 ft wide with 20 ft approaches. All corner radii were less than 20 ft.

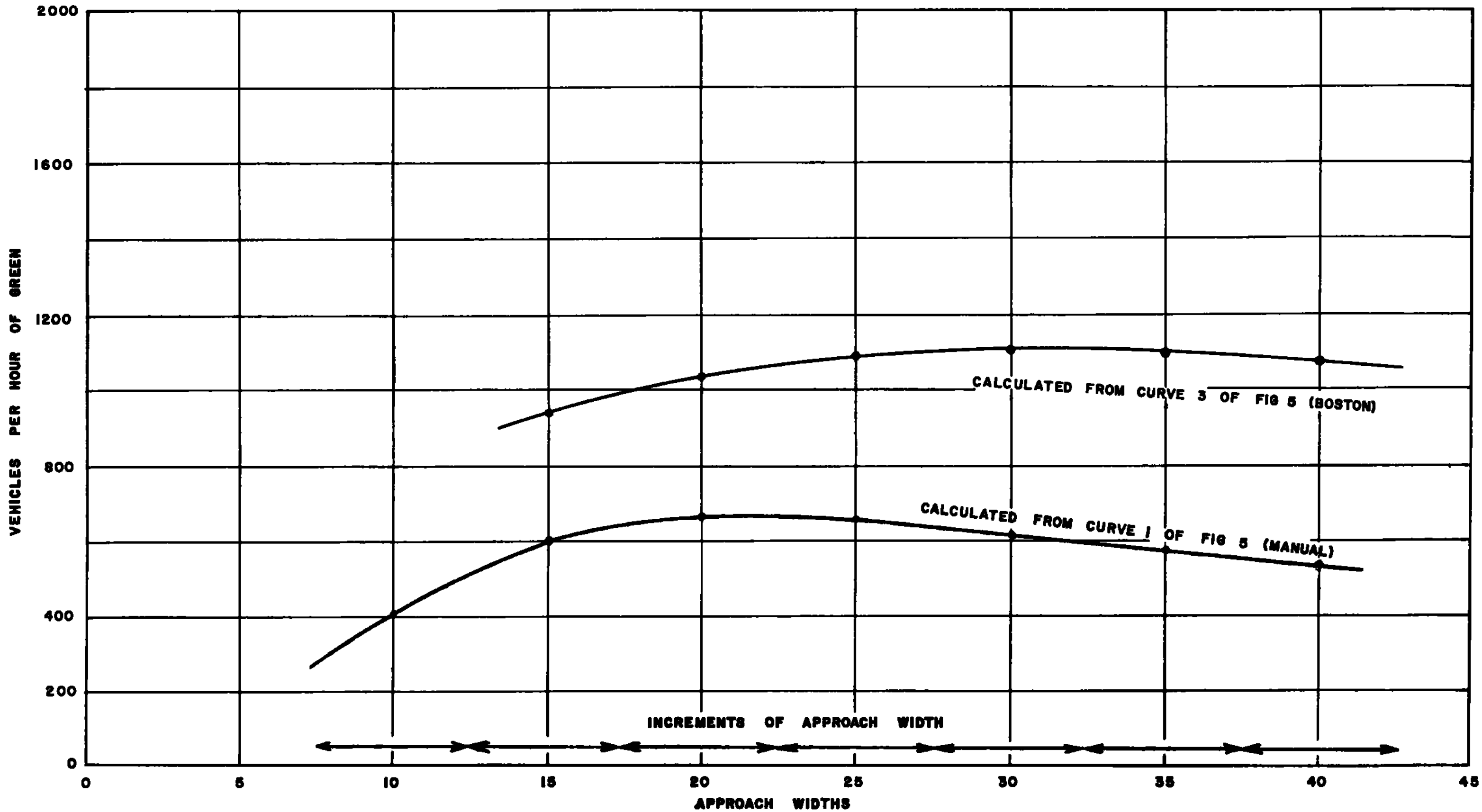


Figure 11. Average reported approach capacities per 10 ft of approach width.

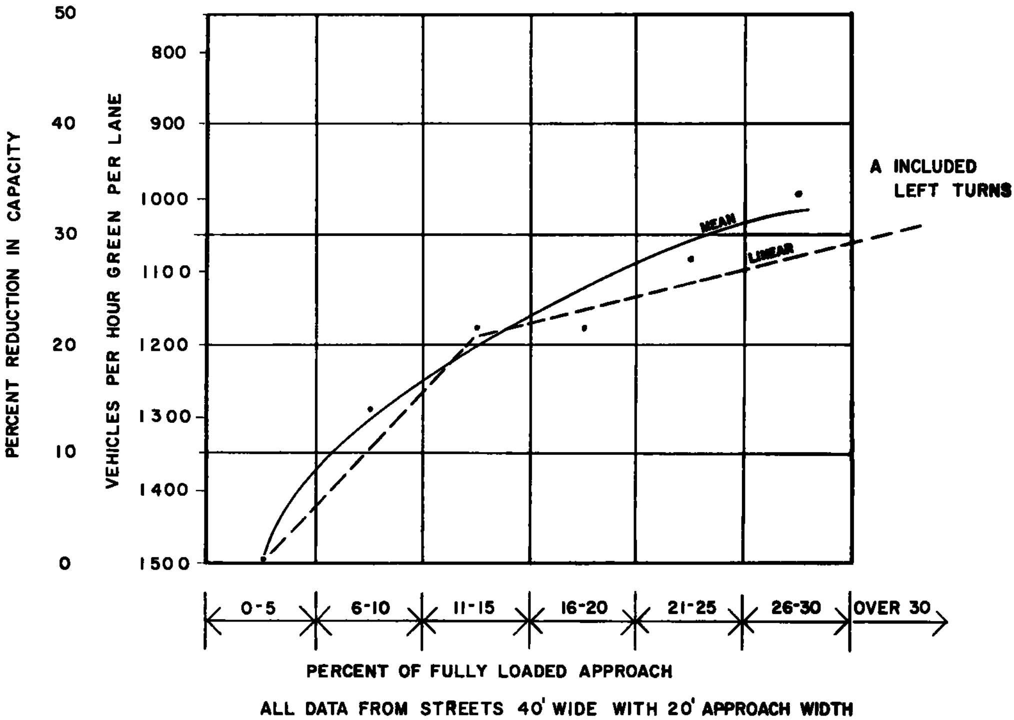


Figure 12. Reductive effect of traffic movements.

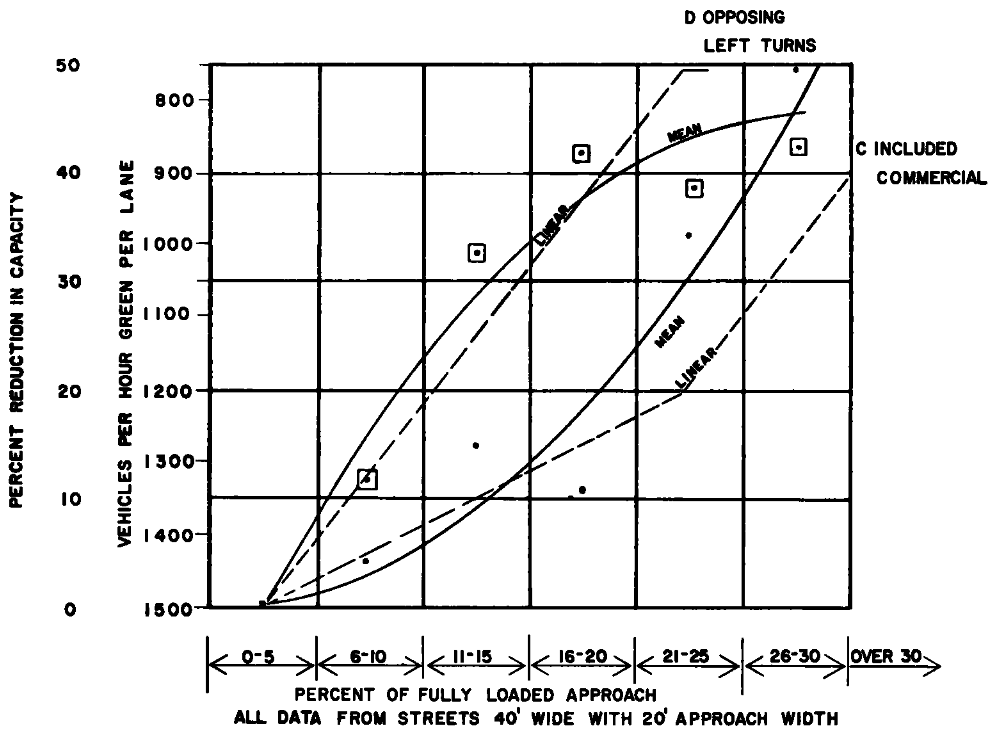


Figure 13. Reductive effect of traffic movements.

After field observations, it was found that the following movements were significantly effective in reducing the capacity of fully loaded approaches: (a) included left turns, (b) included commercial traffic, and (c) opposing left turns.

The first two items were used by the manual, but the last one was not. Buses were too few to be influential in the study. The reductive effects of right turns and total opposing traffic were also included, however the samples available were not sufficient to provide any basis for correlation or linear interpretation.

The study was made by counting traffic volumes, types and directions on each phase of the signal. Counts were made during both morning and evening peaks. Only those phases fully loaded with traffic were recorded. Approximately 2 hours of phase counting was made on each approach.

The data were recorded under each movement as a percentage of the approach volume as shown in Figures 12 and 13. For instance, if the included left turns were 18 percent of the total approach volume, it was listed under the range 16 - 20 percent of fully loaded approach (opposing left turn traffic is expressed in percent of the fully loaded approach).

All data could not be used. For example, if there were 24 percent left turns and 28 percent commercial traffic, it was impossible to measure which item effectively reduced the approach capacity. Therefore, it was decided to select limiting percentages of each movement, and to permit only the movement in question to exceed its limit in the recorded data. This required much tabulation and sorting of data. The limits used are as follows: (a) included left turns, 5 percent; (b) included commercial vehicles, 5 percent; and (c) opposing left turns, 5 percent.

As an example, to measure the effect of a left turn, only those phases were used when the percentage of left turns was high and all other factors were below their limit.

Figures 12 and 13 are mean plots of the summarized data for the selected percentage ranges shown.

An attempt has been made to show the linear interpretation of each plot as follows:

(1) Included left turns — Subtract 2 percent for each 1 percent of left turning traffic less than 10 percent of the total approach, and subtract ½ percent for each 1 percent of left turning traffic over 10 percent of the total approach volume.

(2) Included commercial vehicles — Subtract 1 percent for each 1 percent of commercial traffic less than 20 percent of the total approach, and subtract 2½ percent for each 1 percent of commercial traffic over 20 percent of the total approach volume.

(3) Opposing left turns — Subtract 2½ percent (max. 50 percent) for each 1 percent that the opposing lefts are of the fully loaded approach.

SUMMARY

The preceding study is not an attempt to introduce new theories of intersection capacity measurement, but instead attempts to compare existing local capacities to the average reported capacities of the Highway Capacity Manual.

The comparisons indicate that existing intersection approach volumes, when under pressure, are greater than might be expected. One of the narrowest streets studied, 33 ft wide with 16.5-ft approaches, showed an adjusted capacity of 1,470 vehicles per hour of green as compared to the average reported capacity of 1,030. This is an increase of 43 percent over those calculated from the manual.

Thirteen streets of the common 40 ft width were analyzed and averaged an adjusted capacity of 2,100 vehicles per hour of green, compared to 1,340 as reported in the manual.

Very wide streets of approximately 80 ft widths, with 40-ft approaches indicated adjusted capacities as high as 5,780 vehicles per hour of green compared to reported capacities of 2,160 vehicles per hour of green. The average adjusted capacity of all streets in the range over 65 ft has been found to be 106 percent greater than that reported in the manual.

As outlined, 10 ft is the width deemed standard for one approach traffic lane. It is not uncommon at local intersections to have pressurized traffic form 8 or 9 ft traffic

TABLE I

SUMMATION OF CALCULATIONS

Part A

Approach	Tot. St. Width (Ft.)	Time of Day	Cycle Length (sec)	Signal Phase	Veh/Hr.	% Comm.	% Right Turn	% Left Turn	Bus Stops on Near Side	Combined Adjustment Factor	Sec Green/Cycle	Capacity Load Factor	Approach Width (Ft.)	Veh/Hr Gr Per Approach Width	Veh/Hr Gr Per Tot. Width
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1	64	3-4 pm	90	thru & left	1098	12.3	0	0	No.	1.188	26	1.1	21.5	2900/21.5	3768/64
				right	730	9.5	-	-	No.	1.06	65	1.1	10.5	868/turn	
2	64	3-4 pm	90	thru & left	983	10.0	0	9.9	No.	1.102	26	1.1	21.5	2800/21.5	3075/64
				right	155	5.5	-	-	No.	1.102	51	.9	10.5	275/turn	
3	40	3-4 pm	90	thru & left	895	10.1	0	14.0 (1)	No.	1.08	33	1.1	10.0	2050/10	2250/40
				right	131	25.0	-	-	No.	.892	61	1.1	10.0	200/turn	
4	50	3-4 pm	90	thru & left	646	10.0	0	20.0 (1)	No.	1.05	22	.9	20.0	2800/20	2916/60
				right	115	5.5	-	-	No.	1.102	90	.9	10.0	116/turn	
5	80	3-4 pm	140	full	1653	14.6	11.3	0.4	Yes	.935	67	1.1	40.0	3370/40	3370/80
6	94	7-8 am	140	full	2440	7.6	6.1	1.3	Yes	1.02	67	1.1	47.0	4550/47	4550/94
7	70	7-8 am	140	full	676	22.0	1.3	41.0 (1)	Yes	.70	36	1.1	35.0	3430/35	3430/70
8	80	3-4 pm	140	full	882	35.0	1.6	52.0 (4)	Yes	.77	33	1.1	40.0	4420/40	4420/80

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
9	78	7-8 am	140	full	2248	7.7	0.3	69.5[4]	Yes	.97	51	1.1	39.0	5780/39	$\frac{5780}{78}$
10	40	3-4 pm	78	full	1176	0.6	4.7	18.1	No	1.08	45	.9	20.0	2090/20	$\frac{2090}{40}$
11	40	12-1 pm	78	full	983	0.0	6.2	25.6	No	1.00	45	.9	20.0	1895/20	$\frac{1895}{40}$
12	40	10-11 am	78	full	984	1.1	13.6	1.0	No	1.22	45	.9	20.0	1550/20	$\frac{1550}{40}$
13	33	12-1 pm	78	full	323	15.5	10.8	13.4	No	.95	20	.9	16.5	1470/16.5	$\frac{1470}{33}$
14	40	1-2 pm	76	full	878	0.5	2.3	7.5	No	1.21	40	.9	20.0	1530/20	$\frac{1530}{40}$
15	38	3-4 pm	76	full	1160	0.0	5.7	1.3	No	1.22	50	.9	19.0	1610/19	$\frac{1610}{38}$
16	42	12-1 pm	76	full	405	18.2	18.7	7.2	No	.95	21	.9	21.0	1710/21	$\frac{1710}{42}$
17	40	3-4 pm	40	full	1061	0.3	1.4	2.6	No	1.29	21	.9	20.0	1750/20	$\frac{1750}{40}$
18	40	7-8 am	40	thru & left right	1589 212	0.6 7.6	0.5 -	0 -	No No	1.32 1.075	21 37	.9 .9	10.0 10.0	2540/10 235/turn	$\frac{2775}{40}$
19	50	4-5 pm	40	full	600	5.2	2.7	53.0	Yes	.78	13	1.1	25	2150/25	$\frac{2150}{50}$

TABLE 1 (Continued)

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
20	40	3-4 pm	110	full	1202	8.3	1.6	10.2	No	1.115	75	.9	20	1755/20	1755/40
21	40	7-8 am	110	full	2174	3.6	1.2	7.0	No	1.205	75	1.1	20	2410/20	2410/40
22	40	4-5 pm	110	full	1928	3.2	0.6	10.1	No	1.17	75	1.1	20	2200/20	2200/40
23	40	3-4 pm	110	full	1570	3.9	1.7	11.0	No	1.15	75	.9	20	2220/20	2220/40
24	30	4-5 pm	110	full	551	4.3	83.0	14.5	No	.88	25	1.1	20	2500/20	2500/40
Part B															
25	80	3:30-4 pm [2]		full	866	16.2	10.0	0.0	Yes	.93	810	1.1	40		3760/80
26	80	3-4 pm [3]		full	1596	16.5	10.0	1.0	Yes	.915	1550	1.1	40		3680/80
27	70	3:30-4 pm [2]		full	261	44.5	29.1	3.4 [1]	Yes	.551	436	.9	35		4350/70
28	70	3-4 pm [3]		full	233	50.0	24.2	4.3 [1]	Yes	.515	336	1.1	35		4410/70
29	80	3:30-4 pm [2]		full	435	23.6	5.0	54.0 [4]	Yes	.797	375	1.1	40		4760/80
30	80	3-4 pm [3]		full	748	25.8	3.2	51.8 [4]	Yes	.783	644	1.1	40		4850/80

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
31	40	4-4:30 pm (2)		full	935	0.0	3.0	0.5	No	1.305	1020	1.1	20		2310/40
32	47	3:30-4 pm (2)		full	575	0.5	0.0	3.0	No	1.31	1020	.9	23.5		1720/47
33	47	3-5 pm (3)		full	646	1.0	0.0	4.2	No	1.27	589	1.1	23.5		2820/47

(1) 3 Phase Signal, Treat Left Turns same as Right Turn Adjustments
(2) Peak 1/2 Hour
(3) Using Fully Loaded Phases Per Peak Hour
(4) Because of Intersection Design & Volume Treat Left Turns As Through Traffic

Part A - Using Standard Hourly Traffic Count Data
Part B - Using Special Phase Count Data

lanes. At one approach it was recorded that traffic formed a 6-car approach front on approximately 42 ft of an 80 ft street (Fig. 14). This is an exceptional case.

Further study might prove that specific ranges of approach widths increase greatly in capacity during pressurized congestion, due primarily to the possibility of an extra lane.

An attempt has been made to check the reductive effect of the included commercial traffic and of the various traffic movements. The effective reductions are similar to those presented by the manual. However, the reductive effect of the opposing left turn is probably sufficient to be evaluated alongside the usual lefts, rights and commercial percentages.

Figure 14. An approach loaded to maximum possible capacity. (Note 6 lanes of vehicles)

The basic theory and principles used in the outlined theory of analysis somehow lack certain tangible influences. The manual, under its introduction to signalized intersections, has expressed concern that no existing data is available on certain perlexing factors influencing the capacity. Is it possible that such factors as local highway laws, the degree of highway law enforcement, driver habits, the quality of vehicles, or the degree of over-all congestion have such an effect? No allowance has been made, by either the manual or this study, for side clearances, type of curb, etc. The majority of the intersections in this study are not of a high design, and if additional adjustments providing for poor clearances were made, it is possible that the comparable adjusted capacities would be somewhat higher. The factor having the greatest influence on the capacities found locally was the turning movement. The right turns appeared to cause far less congestion, for a given percent right turning, than the manual indicates.

The indications are that locally 40 percent right turns cause about the same percent reduction (10 percent) in capacity as the 10 percent right turns shown in the manual.

Left turns studied appeared to have a greater initial effect under "pressurized conditions" than the manual indicates. However, this effect appears to lessen on approaches carrying over 20 percent left turning traffic. The local data also indicates that when left turning volumes are equal to, or greater than, 50 percent of the total approach they operate as through traffic with little reductive effect. This is believed to be peculiar to intersections where the major movement is the left turn and the peak condition involves primarily the same daily commuter traffic.

The effect of commercial vehicles was found consistent with that proposed by the manual in the range of 0 to 20 percent. The data indicates a greater effect than that outlined in the manual due to the presence of commercial vehicles when they make up over 20 percent of the total approach volume.

SUMMATION OF CALCULATIONS

All raw data and calculations have been summarized in Table 1. The combined adjustment factor, standardizing traffic to average conditions, is listed under column 10. It is obtained by multiplying together the group of individual adjustments obtained by the traffic characteristics (see sample calculations).

The capacity load factor, indicating if measurements were made under possible or practical conditions, is listed under column 12. A factor of 1.1 indicates that actual field measurements were made under maximum possible capacity condition, whereas 0.9 indicates the traffic count taken during periods of practical capacity condition.

The vehicles per hour of green, comparable to the plot of average reported capacities, is obtained by dividing the volume count by the combined adjustment factor, the capacity load factor, and the seconds of green per seconds of total cycle.

Capacities have been listed for total street widths and for approach widths. Approach widths are equal to one-half of the total street width except for those specific streets where lane marking indicates otherwise. On approaches with separate signal phases, the left-turn movement has been adjusted as right turns, because of the lack of conflicting traffic. Left turn movements were treated as through traffic when in the order of 50 to 70 percent of the total approach volume and protected by signals.

KEY TO APPROACH NUMBERS

		Map Key
1.	Northern Artery northbound at Prison Point Bridge	A
2.	Northern Artery southbound at Prison Point Bridge	A
3.	Prison Point Bridge at Northern Artery	A
4.	Commercial Avenue at Northern Artery	A
5.	Northern Artery northbound at Washington Street	B
6.	Northern Artery southbound at Washington Street	B

KEY TO APPROACH NUMBERS (Continued)

		Map Key
7.	Washington Street westbound at Northern Artery	B
8.	Washington Street eastbound at Northern Artery	B
9.	Northern Artery southbound at Somerville Avenue	C
10.	Memorial Drive westbound at River Street	D
11.	Memorial Drive westbound at River Street	D
12.	Memorial Drive eastbound at River Street	D
13.	River Street southbound at Memorial Drive	D
14.	Memorial Drive westbound at Western Avenue	E
15.	Memorial Drive eastbound at Western Avenue	E
16.	Western Avenue southbound at Memorial Drive	E
17.	Fresh Pond Parkway northbound at Huron Avenue	F
18.	Fresh Pond Parkway southbound at Huron Avenue	F
19.	Huron Avenue eastbound at Fresh Pond Parkway	F
20.	Alewife Brook Parkway southbound at Rindge Avenue	G
21.	Alewife Brook Parkway southbound at Rindge Avenue	G
22.	Alewife Brook Parkway northbound at Rindge Avenue	G
23.	Alewife Brook Parkway northbound at Rindge Avenue	G
24.	Rindge Avenue westbound at Alewife Brook Parkway	G
25.	Northern Artery northbound at Washington Street	B
26.	Northern Artery northbound at Washington Street	B
27.	Washington Street westbound at Northern Artery	B
28.	Washington Street westbound at Northern Artery	B
29.	Washington Street eastbound at Northern Artery	B
30.	Washington Street eastbound at Northern Artery	B
31.	Wm. T. Morrissey Boulevard southbound at Redfield Street	H
32.	Wm. T. Morrissey Boulevard northbound at Redfield Street	H
33.	Wm. T. Morrissey Boulevard northbound at Redfield Street	H

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1. Highway Capacity Manual, Committee on Highway Capacity, Department of Traffic and Operations, Highway Research Board, United States Government Printing Office, Washington, D.C., 1950.

Effect of Temporary Bridge on Parkway Performance

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● THE NEW ENGLAND floods of October 15, 1955, undermined a culvert on the Merritt Parkway at Norwalk, Connecticut, causing a pavement failure. Traffic on the parkway, a principal route between New York and New England, was rerouted with serious congestion and time losses. Because of the urgency of the situation, it was necessary to replace the culvert with temporary structures as rapidly as possible.

The structures used as emergency replacements are shown in Figure 1 as seen by westbound traffic. The bridge in the eastbound roadway is a steel beam span 22 ft wide. In order to conserve time, a pair of bailey bridges were installed in the westbound roadway, one for each lane. The bridge for the outside lane was offset 5 ft because of structural and foundation conditions. The bailey bridges were in use by October 19, four days after the flood.

Shortly after reopening the parkway to traffic, it was observed that the capacity of the bridges in the westbound lane was limited. During several hours of each weekend, traffic volumes were so great as to result in serious congestion, queues of 4 to 5 miles in length frequently being observed during each weekend.

The Bureau of Highway Traffic in cooperation with the Connecticut State Highway Department undertook an analysis of the effects of the temporary bridges on parkway traffic. The purposes of the study were as follows:

1. To determine what factors limited the capacity of the two bridges in the westbound lanes.
2. To analyze the characteristics of a rural freeway traffic stream operating under congested conditions caused by continuing speed-reducing roadway conditions.

STUDY SITE

The profile of the Merritt Parkway in the vicinity of the temporary bridge is shown in Figure 2. The bridge is located in a sag, with sight distance limitations as drivers approach the bridge. Traffic approached the bridge on a 7 percent downgrade and, after crossing the bridge, entered a 5 percent upgrade. There are no horizontal curves through the section shown on the profile. Eastbound and westbound lanes of the parkway are 26 ft wide with mountable curbs and a 21-ft medial divider except at structures. The area immediately preceding the bridge location (station 70 + 00 to station 76 + 00 on the profile) will be referred to as Zone A.

Traffic observed at Zone A was again observed at a point 0.75 mi beyond the bridges. Exit and entrance ramps beyond this point made it inadvisable to make measurements beyond 0.75 mi from the bridge. This second observation station will be referred to as Zone B.

Signing and Speed Zone Program

From station 110 + 00, the nearest parkway entrance before the bridge, there was an extensive sign program directing drivers to "Stay in Line" and informing them of "Temporary Bridge Ahead." The speed limit for the area from station 110 + 00 to the temporary bridge was set at 25 mph. The signs were placed on both the right shoulder and the medial divider at approximately 300-ft intervals as shown in Figure 3.

Bailey Bridge Details

Each bridge in the westbound lane was 80 ft long with a clear distance of 14 ft 6 in. between trusses and a paved roadway width of 12 ft 6 in. The trusses at the outside supports were 3 ft above the roadway and for the inside supports were 8 ft above the roadway. The right lane bridge was offset 5 ft because of structural clearance re-

Figure 1. Temporary bridge site (as seen by westbound traffic).

quirements. A transition zone 100 ft long was paved with asphalt to provide access to the decks of the bridges which were 24 in. above the established grade of the pavement at the bridge site.

COLLECTION OF FIELD DATA

Speed, volume, and headway data for each lane were collected by recording observations on an Esterline-Angus graphic recorder. Observations were made during the hours 4:00 p.m. to 8:00 p.m. on Sunday, July 8, 1956, and from 2:30 p.m. to 6:30 p.m. on Sunday, August 5, 1956. The recorder chart speed was such that time intervals were read to ± 0.02 sec. The pens of the graphic recorder were actuated in two ways. During observations made July 8, the pens were actuated by diaphragms as the

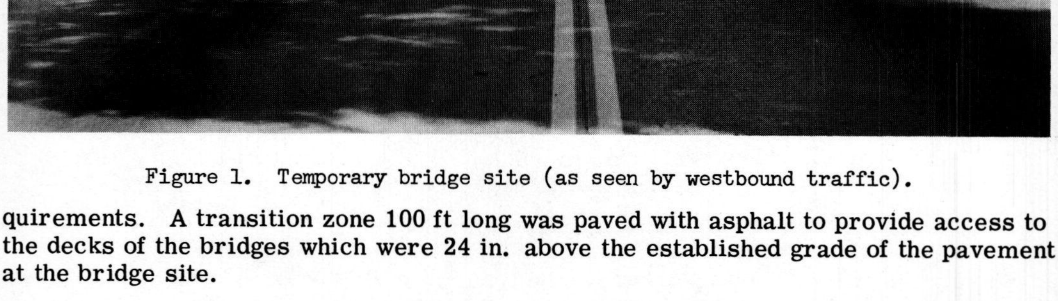

Figure 2. Profile of westbound lanes at temporary bridge site.

Figure 3. Signing program - approaching temporary bridge.

vehicles passed over air-hoses stretched across the roadway. To eliminate any adverse effect that the hoses would have on slow-moving traffic, the second group of observations on August 5 were made with manual actuation of the pens. Subsequent analysis of the data showed there was no significant difference in data collected by the two methods. Speeds were determined by finding the travel time between successive

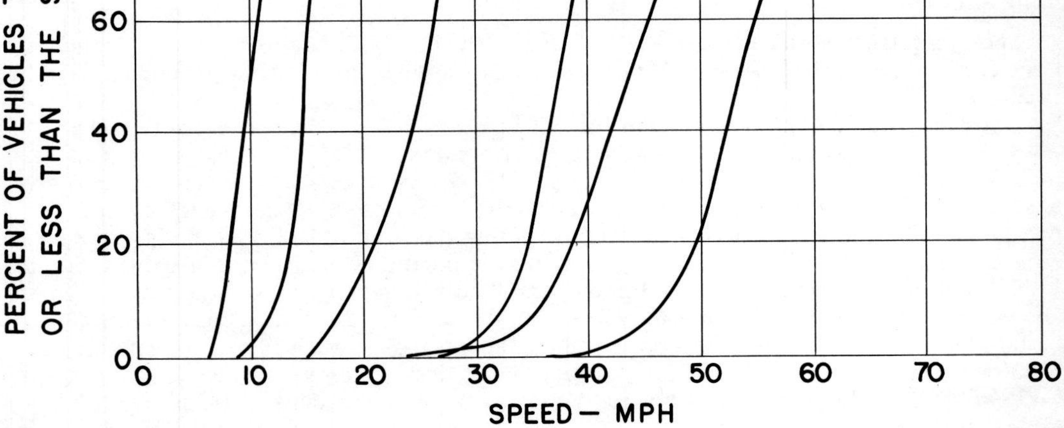

Figure 4. Speed distribution - Zones A and B.

TABLE 1

SUMMARY OF VOLUME, SPEED, DENSITY, HEADWAY, AND SPACING
INSIDE LANE - ZONE A

Date 1956	Time, p.m.	Volume, vph	Speed, mph	Density, veh/mi	Mean Headway, sec	Mean Spacing, ft
July						
8	5:00-5:05	876		No Record		
	5:15-5:20	1,044		No Record		
	5:40-5:45	1,044	7.8	133.8	3.45	39.5
	5:55-6:00	1,212	9.0	134.5	2.97	39.3
	6:15-6:20	876	8.1	108.2	4.11	48.8
	6:25-6:30	1,092	8.4	129.8	3.30	40.7
	6:35-6:40	1,116	9.9	112.4	3.23	47.0
	6:55-7:00	1,116	10.5	106.5	3.23	49.5
	7:05-7:10	1,080	9.2	117.7	3.33	44.9
	7:15-7:20	1,104	9.4	117.8	3.27	44.9
	7:25-7:30	1,128	9.4	121.0	3.19	43.6
	7:35-7:40	1,116	8.4	132.2	3.23	40.0
Aug.						
5	2:50-2:55	1,320	26.4	50.0	2.73	105.6
	2:55-3:00	1,032	25.7	40.2	3.49	131.5
	3:00-3:05	912	30.5	29.2	3.95	176.6
	3:05-3:10	1,020	21.9	46.6	3.53	113.2
	3:20-3:25	1,128	27.4	41.1	3.19	128.4
	3:37-3:42	1,068	10.5	101.9	3.37	51.8
	5:00-5:05	1,176	11.0	107.1	3.06	49.3
	5:10-5:15	1,164	12.0	96.8	3.09	54.5
	5:20-5:25	1,104	10.1	109.1	3.26	48.4
	5:49-5:54	1,128	12.4	91.3	3.19	57.8
	5:59-6:04	1,068	10.5	101.5	3.37	51.9
	6:09-6:14	1,140	12.1	94.6	3.16	55.8

points on the pavement. The lengths of the speed traps varied from 50 ft to 126 ft, as determined by location of pavement joints and traffic conditions. The same method of observation was employed at Zone A and at Zone B, although no attempt was made to identify individual vehicles.

Traffic Conditions

The Merritt Parkway is a limited-access, 4-lane, divided facility for passenger cars only. Generally, the speed limit, except as applied to the area preceding Zone A, is 55 mph.

Two vehicle actuated detectors of the overhead radar type had been previously installed over both westbound lanes at a point ¼ mi west of Zone A. The number of vehicles passing the detectors in each 2-min. period was automatically converted to an hourly volume rate, the results being placed on a graphic-recorder chart so that a continuous record of volume was available during the time of the field study. Total volume rates for both westbound lanes during the period of study ranged from 1,500 to 2,500 vehicles per hour. Peak volumes remained substantially constant between 2,000 and 2,250 vph for the two westbound lanes combined from 3:00 p.m. until 10:00 p.m. on both days that field work was conducted. During these hours the temporary bridges were operating at capacity with long queues of waiting vehicles.

Preliminary observations of traffic during periods of low density (less than 40 vehicles per mile) at Zone A indicated that drivers applied their brakes while still on the downgrade and continued to apply brakes until they passed the sag point in the vertical curve. Vehicles began to accelerate before reaching the transition pavement at the bridges. Maximum density of vehicular traffic occurred in the area from 100 to 600 ft before the bridge, so that measurements of volume, speed and density were made in this area rather than on the bridge itself.

Accident Experience

A complete record of type and severity of accident experience at the bridge was not

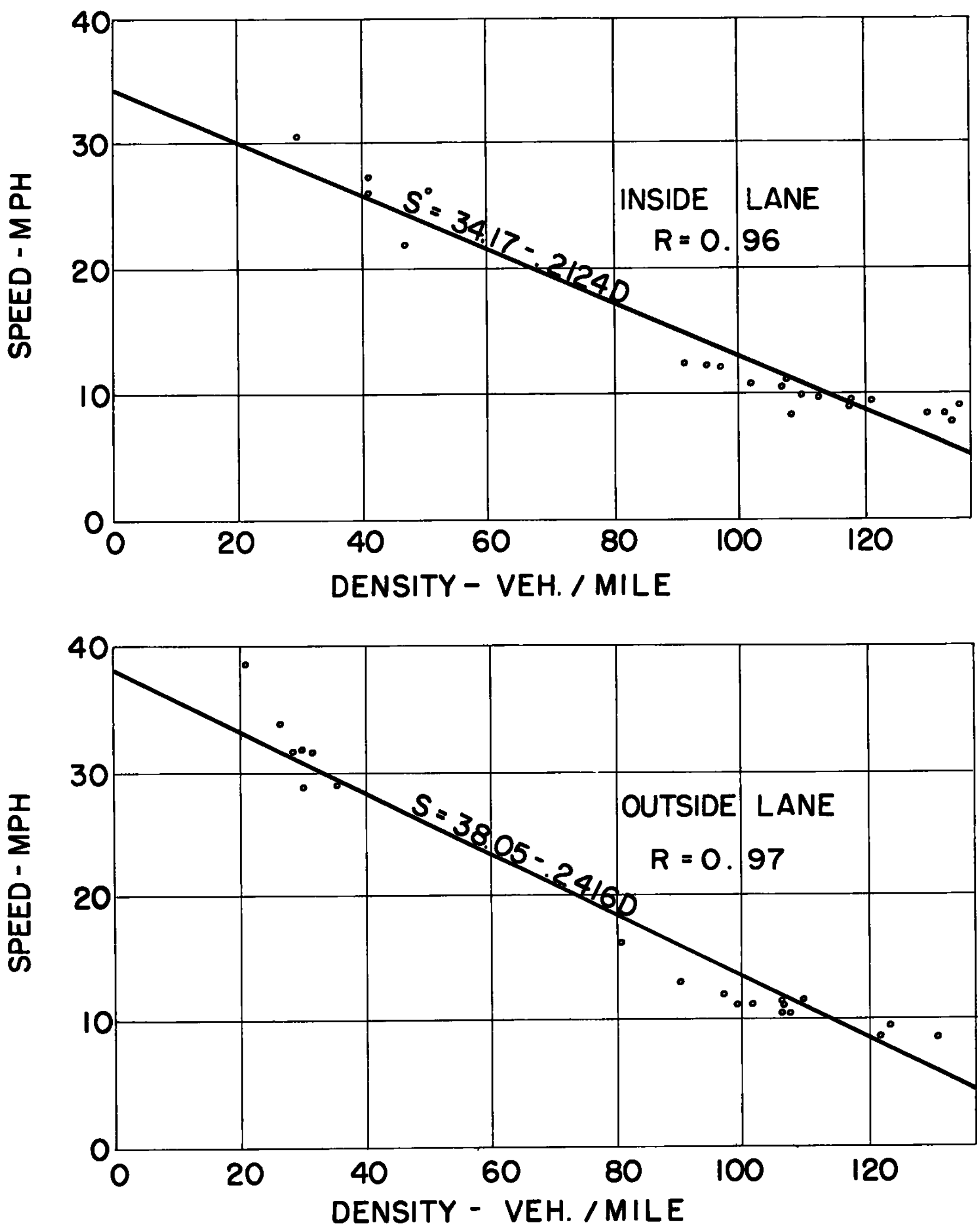

Figure 5. Speed versus density.

TABLE 2

SUMMARY OF VOLUME, SPEED, DENSITY, HEADWAY, AND SPACING
OUTSIDE LANE - ZONE A

Date 1956	Time, p.m.	Volume, vph	Speed, mph	Density, veh/mi	Mean Headway, sec	Mean Spacing, ft
July						
8	5:00-5:05	792	38.8	20.4	4.54	258.9
	5:15-5:20	864	31.5	27.4	4.17	192.5
	5:40-5:45	1,128	10.6	106.2	3.19	49.7
	5:55-6:00	1,296	16.1	80.4	2.78	65.7
	6:15-6:20	1,092	7.7	141.3	3.30	37.4
	6:25-6:30	1,092	8.3	130.9	3.30	40.3
	6:35-6:40	1,032	8.5	121.7	3.49	43.4
	6:55-7:00	1,176	11.1	106.5	3.06	49.5
	7:05-7:10	1,128	8.6	130.5	3.19	40.4
	7:15-7:20	1,116	11.1	101.1	3.23	52.2
	7:25-7:30	1,212	9.8	123.9	2.97	42.6
	7:35-7:40	1,128	7.8	144.2	3.19	36.6
Aug.						
5	2:50-2:55	936	31.8	29.5	3.85	179.3
	2:55-3:00	972	31.6	30.8	3.70	171.5
	3:00-3:05	900	34.0	26.5	4.00	199.2
	3:05-3:10	1,032	28.9	35.7	3.49	148.0
	3:20-3:25	864	28.8	30.0	4.17	176.0
	3:37-3:42	1,116	10.5	106.2	3.23	49.7
	5:00-5:05	1,188	12.3	97.0	3.03	54.4
	5:10-5:15	1,188	13.2	90.1	3.03	58.6
	5:20-5:25	1,212	11.4	106.7	2.97	49.5
	5:49-5:54	1,116	11.2	99.3	3.23	53.2
	5:59-6:04	1,104	10.3	107.2	3.26	49.2
	6:09-6:14	1,248	11.4	109.1	4.53	48.3

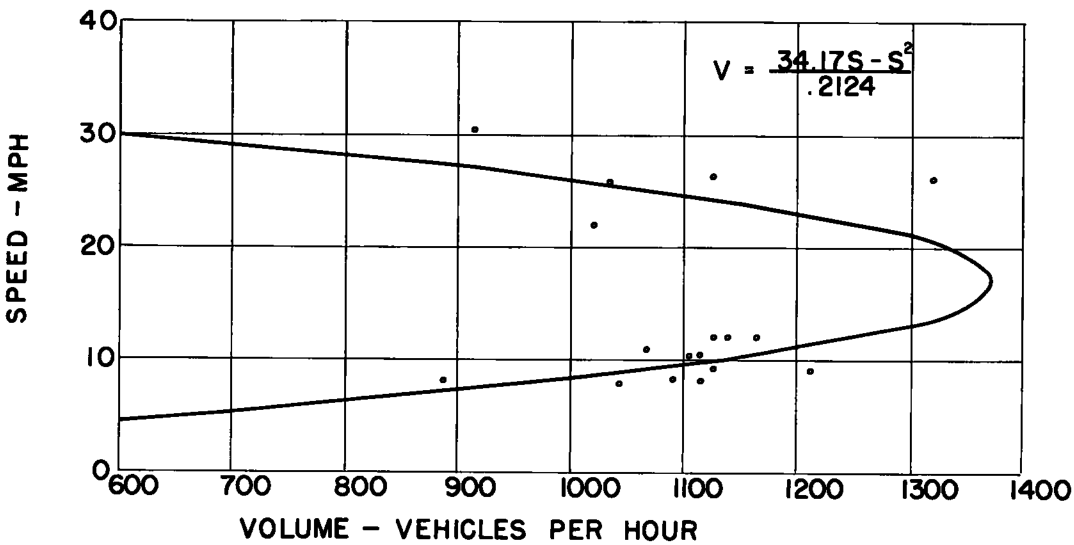

Figure 6. Speed versus volume - inside lane.

available at the time this paper was prepared although the number of accidents is known. During the period the bridges were in operation, October 1955 to October 1956, there were 3 accidents involving the bridge. Two of these were caused by sleeping or inattentive drivers and the other accident was caused by a driver, whose brakes had failed, ramming the bridge in order to come to a stop.

In the 2,000 ft preceding the bridge, there were 7 accidents, the majority being rear-end collisions involving two or three vehicles. The extensive signing program apparently alerted the drivers to the unusual conditions in the roadway and kept the accident frequency to a minimum.

ANALYSIS

In the following analysis various 5-min. volume counts have been expanded to an

TABLE 3

SUMMARY OF VOLUME, SPEED, DENSITY, HEADWAY, AND SPACING INSIDE LANE - ZONE B

Date 1956	Time, p.m.	Volume, vph	Speed, mph	Density, veh/mi	Mean Headway, sec	Mean Spacing, ft
July 8	4:10-4:15	1,056	48.9	21.6	3.41	244
	4:25-4:30	1,230	46.4	26.5	2.93	199
	5:00-5:05	1,573	43.1	36.5	2.29	145
	5:14-5:19	1,254	45.8	27.4	2.87	193
	6:00-6:05	760	47.7	16.0	4.73	331
	6:15-6:20	1,244	43.2	28.8	2.89	183
	6:27-6:32	1,210	42.9	28.2	2.98	187
	7:16-7:21	1,334	44.1	30.3	2.70	174
	7:25-7:30	1,334	46.2	28.9	2.70	183
	7:45-7:50	1,329	48.9	27.2	2.71	194
	8:00-8:05	1,376	44.6	30.9	2.62	171
Aug. 5	2:40-2:45	1,355	56.6	23.9	2.66	221
	3:00-3:05	1,176	54.8	21.4	3.06	247
	3:45-3:50	1,176	52.5	22.4	3.06	235
	3:55-4:00	1,411	51.8	27.2	2.55	194
	4:05-4:10	1,317	50.5	26.1	2.73	202
	4:50-4:55	1,341	51.2	26.2	2.69	202
	5:00-5:05	1,388	44.1	31.5	2.59	168
	5:10-5:15	1,235	43.8	28.2	2.92	187
	5:35-5:40	1,211	47.0	25.8	2.97	205
	5:45-5:50	1,329	48.1	27.7	2.71	191

equivalent number of vehicles per hour. Mean speed, density and spacing were also calculated for each of these 5-min intervals. All mean speed calculations are based on space-mean speed according to the following formula:

$$\bar{S} = \frac{(L)\ (n)}{1.47\,\Sigma t} \qquad (1)$$

in which

$\bar{S}$ = space mean speed of all vehicles in mph;
L = length of trap in feet;
n = number of observed vehicles;
Σt = total time in seconds for passage of n vehicles through trap; and
1.47 = conversion factor (ft per sec to mph).

Wardrop (1) has shown that theoretical calculations involving volume, speed, and density relationships hold true only when the space mean speed is used.

In the following discussion the inside lane will refer to the left lane nearest the medial divider, and the outside lane will refer to the right lane as seen by the driver.

Distribution of Speeds

A comparison of vehicle speeds at Zone A with speeds at Zone B is shown in Figure 4. Curves A and B are representative speed distributions of the inside lane and outside lane, respectively, at Zone B. The speed limit in this zone is 55 mph. The mean speed of the inside lane is 53.9 mph and of the outside lane is 43.9 mph. Although vehicles may have not yet attained their maximum speed at Zone B, the curves shown here agree very well with the results of recent studies made by the Connecticut State Highway Department on the Merritt Parkway.

TABLE 4

SUMMARY OF VOLUME, SPEED, DENSITY, HEADWAY, AND SPACING
OUTSIDE LANE - ZONE B

Date 1956	Time, p.m.	Volume, vph	Speed, mph	Density, veh/mi	Mean Headway, sec	Mean Spacing, ft
July						
8	4:10-4:15	859	43.1	19.9	4.19	265
	4:25-4:30	894	41.8	21.4	4.03	247
	5:00-5:05	946	41.4	22.9	3.80	231
	5:14-5:19	832	43.6	19.1	4.33	277
	6:00-6:05	703	44.1	16.0	5.12	331
	6:15-6:20	991	40.9	24.2	3.63	218
	6:27-6:32	899	41.4	21.7	4.01	243
	7:16-7:21	775	42.3	18.3	4.64	288
	7:25-7:30	901	44.7	20.2	3.97	262
	7:45-7:50	929	45.0	20.6	3.87	256
	8:00-8:05	870	42.1	20.7	4.14	253
Aug.						
5	2:40-2:45	1,000	42.9	23.2	3.60	228
	3:00-3:05	835	48.0	17.4	4.31	303
	3:45-3:50	894	46.3	19.3	4.03	273
	3:55-4:00	870	46.3	18.8	4.14	281
	4:05-4:10	858	43.4	19.8	4.19	267
	4:50-4:55	847	46.8	18.1	4.25	292
	5:00-5:05	906	42.0	21.6	3.98	245
	5:10-5:15	847	42.5	19.9	4.25	265
	5:35-5:40	870	44.3	19.6	4.14	269
	5:45-5:50	964	44.8	21.5	3.73	245

The speed distribution of uncongested traffic flow (density less than 40 vehicles per mile) in the outside lane at Zone A is shown in curves C and D. Curve C represents the speed of vehicles at a point 560 ft before the bridge, and curve D represents speeds at a point 150 ft before the bridge. The mean speed for curve C is 37.9 mph and for curve D is 25.7 mph. The speed limit was 25 mph through this zone. As vehicles continued beyond the 150 ft point, the drivers maintained or increased speed slightly in crossing the bridge.

The combined effect of congestion and the bridge is shown in curves E and F. Under these conditions minimum speeds were observed 560 ft before the bridge, and drivers were accelerating when passing the point 150 ft before the bridge. Curve E with a mean speed of 10.3 mph represents the distribution of vehicle speeds 560 ft before the bridge, and curve F with a mean speed of 15.5 mph represents the distribution of vehicle speeds 150 ft before the bridge. Since minimum speeds were observed 560 ft before the bridge, all observations for Zone A were made at this point.

Vehicular Volumes and Lane Distribution

The hourly rate of volumes as estimated from 5-min periods for Zone A are shown in Tables 1 and 2 and for Zone B in Tables 3 and 4. An examination of volumes at Zone A indicates that when mean speeds exceed 20 mph and densities are less than 50 veh per mi, a greater percentage of the vehicles use the inside lane than the outside lane. The mean hourly volume rate, during those 5-min periods that satisfy the above conditions of speed and density is 1,936 vph for both lanes. Of these, an average 1,047 vph (54 percent) use the inside lane.

With increasing congestion, as measured by speed and density, the volumes by lanes become more nearly equal. Considering the volumes after 5:40 p.m., July 8, and after 3:37 p.m., August 5, the mean hourly volume was 2,253 vph for both lanes. Of these, an average of 1,102 vph (48.9 percent) used the inside lane and 1,151 vph used the outside lane.

A comparison of the estimates of hourly volume rates for the pair of lanes shows a significant difference in capacity of the two lanes at the 5 percent confidence level. If the period 6:15 p.m. - 6:20 p.m., July 8, is eliminated from the comparison, there is no longer a significant difference between the volumes of the two lanes. It is probable that some stoppage occurred beyond the limits of the study that reduced the volumes using the inside lane during the 6:15 p.m. - 6:20 p.m. period.

The lane distribution at Zone B shows a different pattern from that at Zone A. In this instance the average 5-min estimate of hourly volume for both lanes is 2,149 vph, of which 1,269 vph (59 percent) use the inside lane and 880 vph use the outside lane. Many drivers were observed to be making passing maneuvers in this area since passing had been restricted for several miles in the long queues delayed by the temporary bridge.

Speed, Volume, Density Relationships

The Highway Capacity Manual (2) points out that maximum volumes for a given facility occur at an optimum speed and that above and below this optimum speed there will be lesser volumes. Greenshields (3) showed that there was a straight line relationship between speed and density of traffic. Forbes (4) and Greenshields have shown that the relationship between volume and speed can be fitted with a parabolic curve. These relationships were applied to the data obtained at Zone A.

Hourly volume rates, space-mean speed, density, average headway and spacing of vehicles were computed for each 5-min sample (Tables 1-4). Density was found by dividing the hourly volume by the mean speed. Headways and spacings are based on averages for all of the vehicles in each 5-min sample.

The various points obtained for speed and density were plotted and straight lines fitted to them by the method of least squares. Each 5-min point was given equal weight in computing the least squares. The curves, one for each lane, are shown in Figure 5. The regression equation for the inside lane at Zone A is given by:

$$S = 34.17 - 0.2124\,D; \qquad (2)$$

and for the outside lane by:

$$S = 38.05 - 0.2416\,D \qquad (3)$$

in which

S = speed in miles per hour, and
D = density in vehicles per mile.

In the above equation 38.05 and 34.17 represent the speed as vehicles approached the bridge without congestion. These two values would be represented by the mean speed of curve C (Fig. 4) for which the mean speed was 37.9 mph. The coefficient of correlation for the inside lane is 0.97 and for the outside lane, 0.96.

The nature of the relationship between speed and volume follows from the relationship between speed and density. The above equations are of the form $S = a - mD$ and,

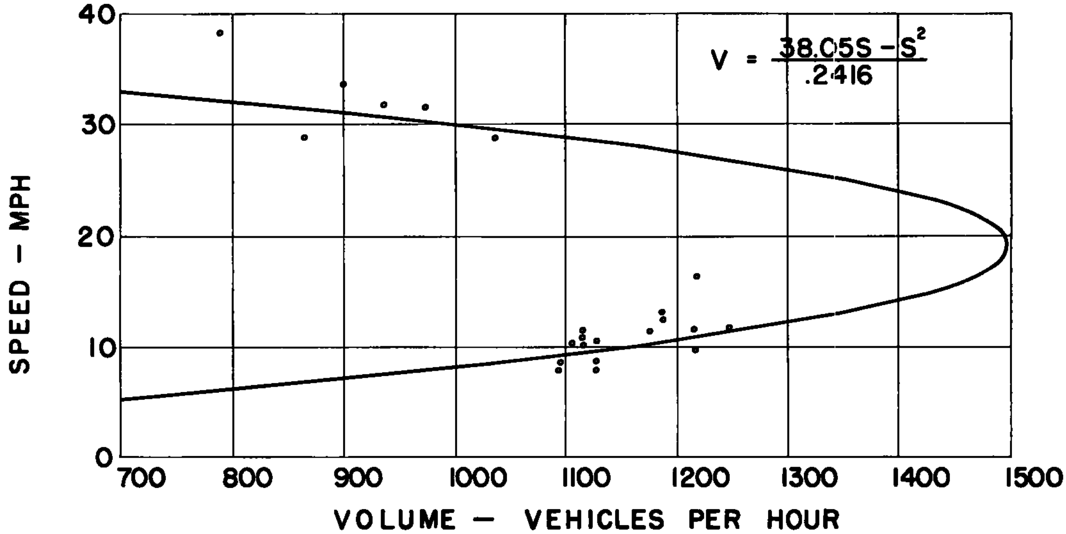

Figure 7. Speed versus volume - outside lane.

since $D = V/S$, the equation may be written as

$$S = a - m\,V/S \tag{4a}$$

or

$$V = \frac{aS - S^2}{m} \tag{4b}$$

in which

V = volume in vehicles per hour, and
S = space-mean speed in mph.

There are two values of speed which will satisfy a given volume.

The equation for the speed-volume relationship of the inside lane at Zone A is

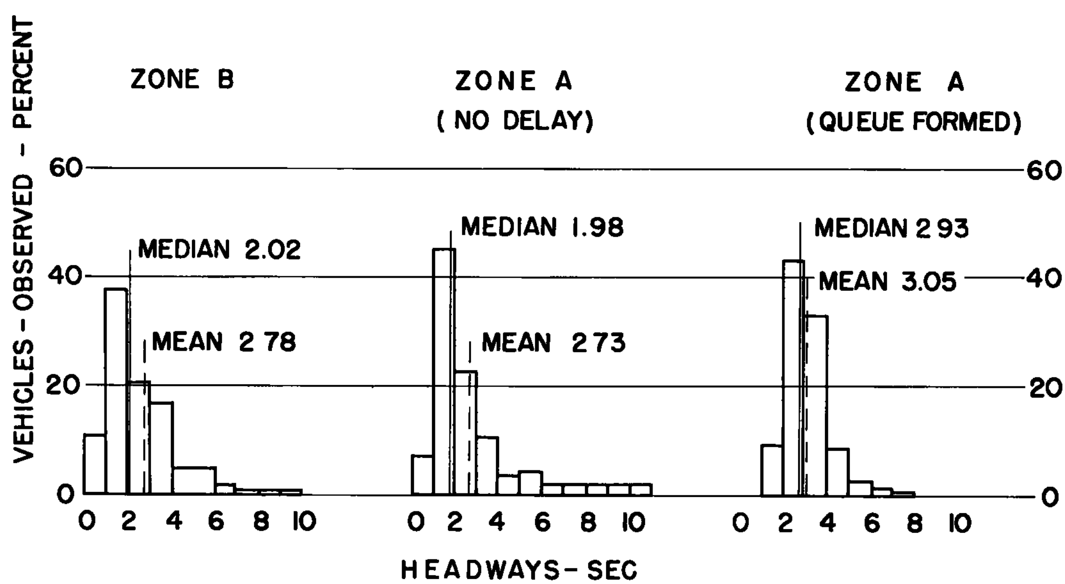

Figure 8. Headway distributions.

given by:

$$V = \frac{34.17\,S - S^2}{0.2124} \tag{5}$$

and for the outside lane by:

$$V = \frac{38.05\,S - S^2}{0.2416} \tag{6}$$

The curves of these two equations are plotted in Figures 6 and 7.

Under the existing roadway and traffic conditions, the possible capacity is represented by the apex of the curve. The critical speed, at which the maximum volume will occur, is 17 mph and 19 mph for the inside and outside lanes respectively. The critical densities corresponding to these critical speeds are 80.4 veh per mi and 78.7 veh per mi for the inside and outside lanes. The possible capacity of the inside lane is 1,375 vph and for the outside lane, 1,498 vph. The possible capacity as used here differs from the Highway Capacity Manual in that a lower operating speed is considered. Maximum observed volume for the outside lane was 1,248 vph and for the inside lane, 1,320 vph.

As volumes increased and speeds decreased (the upper limb of the parabola) the density also increased. Curve D (Fig. 4) showed that under conditions of free flow 15 percent of the vehicles were traveling at less than the critical speed of 19 mph. As the volume (and density) of the traffic increased, the mean speed decreased and increasing numbers of vehicles were operating at less than the critical speed. The volumes occurring at those points at which the mean speed fell below the critical speed are plotted on the lower limb of the parabolic curve.

The curves also indicate that if the volume per lane approaching the bridges had not exceeded 1,000 to 1,100 vph, the vehicles could have continued through the test section at speeds between 20 and 30 mph. As volumes per lane approaching the bridge exceeded 1,100 vph the average speed quickly dropped below the critical speed and, although greater volumes passed over the bridges, it was accomplished at the expense of considerable delay and congestion to the individual drivers.

Vehicle Spacing and Headways

Mean spacing, in feet, between vehicles in the same lane for each 5-min period was calculated from the relationship:

$$\text{Spacing} = \frac{5,280}{\text{Density (veh/mi)}} = 1.47 \text{ space-mean speed (mph) x mean headway (sec)} \tag{7}$$

Resulting values are found in Table 1 through Table 4. The minimum average spacing was 36.6 ft during the 5-min period beginning at 7:35 p.m., July 8, in the outside lane. Average spacings for those volumes on the lower limb of the curves (congested conditions) varied from 36.6 ft at a mean speed of 7.8 mph to 65.7 ft at a mean speed of 16.1 mph. The values of average spacing are in general agreement with extrapolated values of minimum spacings at various speeds as shown in Figure 2, page 28 of the Highway Capacity Manual.

The influence of congestion on headways is indicated in Figure 8. Headways shown are on a per lane basis. Each of the three distributions is based on a minimum of three 5-min samples. The headway distribution at Zone B is based on a volume of 1,290 vph per lane; at Zone A (no delay) on a volume of 1,320 vph per lane; and at Zone A (queue formed) on a volume of 1,190 vph per lane.

When traffic is not congested 48 percent of the headways at Zone B and 52 percent of the headways at Zone A (no delay) are less than 2 seconds. These values are in close agreement with the theoretical distribution as determined from the Poisson Series. The theoretical distribution of headways less than 2 seconds are 51.2 percent and 51.8 percent for Zone B and Zone A (no delay), respectively.

As density increased (and spacing decreased) the vehicles were required to maintain greater headways than normally expected. The effect on headways less than 2 seconds is shown by the histogram for Zone A (queue formed) in Figure 8. The theoretical distribution as determined from the Poisson Series indicates an expected value of 48.2 percent headways less than 2 seconds. Observed headways less than 2 seconds at Zone A (queue formed) were 9.8 percent of the total headways. The requirement that drivers maintain a minimum spacing would restrict the number of vehicles observed traveling at less than 2 seconds headway. The failure to maintain headways under 2 seconds increases the mean headway and limits the capacity at the bridges.

SUMMARY

1. The average speed of vehicles during uncongested periods was 37.9 mph at a point 560 ft before the bridge and 25.7 mph at a point 150 ft before the bridge. As density increased the mean speed of vehicles was 10.3 mph at 560 ft before the bridge and 15.5 mph at 150 ft. Mean speeds 0.75 mi beyond the bridge were 53.9 mph and 43.9 mph for the inside and outside lanes, respectively.
2. The mean estimate of hourly volumes passing the bridge during congested periods was 2,253 vph for both lanes. An average 1,102 vph (48.9 percent) used the inside lane and 1,151 vph, the outside lane. Under conditions of lesser congestion 54 percent of the vehicles used the inside lane at a total volume of 1,936 vph for both lanes.
3. As density increases, speed decreases in a straight-line relationship. The equation of the least-squares line for the inside lane is given by $S = 34.17 - 0.2124\ D$ and for the outside lane by $S = 38.05 - 0.2416\ D$. Coefficients of correlation were 0.96 and 0.97 for the inside and outside lanes.
4. Speed decreased as volume increased up to a point of critical speed and corresponding critical density. Below the critical speed a further decrease in speed resulted in a further decrease in volume. The relationship can be described by a parabolic curve the apex of which represents the possible capacity at the critical speed. Points were observed and plotted on both limbs of the parabola.
5. Minimum average spacings as low as 36.6 ft at 7.8 mph were observed. The requirement of maintaining a minimum spacing at low speeds resulted in a decrease in the number of headways under 2 seconds. This in turn increased the mean headway and consequently limited the volumes observed at the bridge.

ACKNOWLEDGMENT

The field studies and the preparation of charts were carried on by the Connecticut State Highway Department.

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Trends in the 30th-Hour Factor

William P. Walker, Highway Transport Research Engineer, Bureau of Public Roads

An analysis of automatic traffic recorder data for rural highways reveals that the 30th-hour factor exhibits a tendency to decline slightly with the passing of time, rather than to remain stable as past indications have suggested. Records for 160 traffic-recorder stations in continuous operation from 1956 through 1953 provided the basic data for the analysis. All classes of rural highways were represented and the coverage included 26 states. The average factor for these stations declined at the average rate of 0.11 per year over the 7-yr period but a wide variation in the rate of decline was found between different stations. Roads with volumes of more than 3,000 vehicles ADT experienced a more rapid rate of decline in the factor than the roads with lesser volumes. Also, factors of 15, or greater, experienced a more rapid rate of decline than factors of less than 15. A table which relates the annual change in the factor to both the magnitude of the factor and the ADT is included in the report.

● IN 1940 an investigation was made of the relation between traffic volumes during peak hours and the annual average daily traffic volumes on a number of rural highways. Records from automatic traffic recorders provided the basic material for this investigation. Comparatively few of these recorders were in use at that time and none had been in continuous service for longer than 3 or 4 years. Nevertheless, a striking correlation was found between peak-hour volumes and the average daily traffic on rural highways. The authors of a report (1) on these investigations recommended that highways be designed to accomodate a volume of traffic at least as great as that which would occur during the 50th highest hour of the year, but no greater than that for the 30th highest hour. The American Association of State Highway Officials adopted the policy that highways should be designed for the 30th highest hourly volume of the year for which the highway was being built.

The work was reviewed in 1945 when better counter coverage had been established and the period of continuous operation had been extended. The results of this review (2) strengthened the recommendations of the 1940 study, and drew the additional conclusion that for any particular facility the ratio of the 30th highest hourly volume to the average annual daily volume changed very little, if at all, from year to year. This ratio, normally expressed as a percentage figure, is often referred to as the 30th-hour factor. It is a coefficient which, if known for a particular road, will yield the 30th highest hourly volume when applied to the average annual daily traffic on that road.

The present reexamination of traffic data is for the purpose of detecting any trends that may exist in the magnitude of the 30th-hour factor. If the factor for any road is indeed fixed or stable, as indications in the past have suggested, then a means is assured for estimating design-hour volumes with a degree of confidence as great as that for the estimate of the average daily traffic. If there is any tendency for the factor to become either larger or smaller with the passing of time, then the rate of change should be determined so that appropriate adjustment can be made in the design-hour volume for any future year. Unless proper adjustment of the factor is made, facilities designed for future traffic will be either overdesigned for their traffic load or they will become congested in a shorter period of time than anticipated, even though the future daily traffic is accurately predicted.

RECORDS COVER 7-YEAR PERIOD

Records for 160 counters that were in continuous operation during the period 1946-53 are used in this analysis. All of these counters were located on rural highways and covered 26 states distributed from coast to coast. All classes of rural roads were represented and the range in traffic volume was from a few hundred vehicles per day on the most lightly traveled roads to more than 25,000 vehicles per day on the more heavily traveled multi-lane roads.[1]

Wide Variation Between Factors at Different Locations

For the year 1946, the 30th-hour factors for the 160 locations ranged in magnitude from less than 9.0 to above 30.0. The frequency of occurrence of factors of various magnitudes is shown in Figure 1 for 2 years, 1946 and 1953. The similarity between the distributions of factors for the 2 years is very striking. The distribution curve for the year 1953 is located slightly to the left of the one for 1946 and this is an indication that the factors for the more recent year are somewhat smaller than they were in 1946. The fact that a reduction in the factor has occurred during the 7-yr period is more apparent in Figure 2. This figure shows the cumulative number of traffic counter locations for which the 30th-hour factors are equal to or less than the values specified. Data for 1946 and 1953 are again represented.

A fact not shown by Figure 2 is that factors for some stations changed by a far greater amount than others. Forty-six of the 160 stations actually experienced an increase in the factor during the 7-yr period. For the remaining 114 stations (71 percent of the total) the factor showed a decline. The average factor changed from 14.07 in 1946 to 13.25 in 1953, a decrease of 0.82.

Greatest Changes on High-Volume Roads

On the average, the roads carrying relatively low volumes (below 2,000) had the highest factors, and those carrying the heaviest volumes had the lowest factors. This is illustrated in Figure 3, where the stations have been divided into three volume groups namely, those below 2,000, those between 2,000 and 3,000, and those over 3,000 ADT. These were the volumes in 1951.

Roads within the volume range of 2,000 to 3,000 vehicles per day had the least change in the factor between 1946 and 1953. In 1946 the factor for this group was 13.8 as compared with 13.6 in 1953. The trend throughout the 7-yr period was far from uniform and it is questionable whether the 18 stations in this group constitute a sufficient sample to justify conclusions. The sample is not only small but is biased in favor of stations having low factors.

For roads having a volume of 2,000 vehicles or less, the average factor declined from 16.2 in 1946 to 15.4 in 1953, a change of -0.8. There were 43 stations in this group. The group of roads having an average daily volume of over 3,000 showed the greatest change of all the groups. For these roads the average factor was 13.1 in 1946 and 12.0 in 1953. The difference between the factors for these years is 1.1. A total of 99 stations comprise this group and the trend over the years appears to be very consistent.

The curves in Figure 3 suggest that for roads carrying between 2,000 and 3,000 vehicles per day, the change in the factor with the passage of time is almost nil. As a class, roads carrying fewer than 2,000 vehicles ADT have shown a decline in the 30th-hour factor of about 0.11 per year. For roads carrying more than 3,000 vehicles ADT, the factor has shown the rather marked rate of decline of 0.16 per year.

Least Changes on Roads with Low Factors

If the apparent finding as stated above were applied indiscriminately, the 30th-hour factors for the most heavily traveled roads would rapidly be approaching the absolute minimum, which is theoretically 4.15, assuming the traffic volume is constant during all hours of the year. None of the stations in the sample had a factor anywhere near

[1] The traffic counter stations used in this analysis are listed according to state identification number in an appendix to this report. Individuals or agencies having a bona fide interest may obtain detailed traffic data for any station or group of stations identified in this list by addressing the U.S. Bureau of Public Roads, Highway Transport Research Division, Washington 25, D.C.

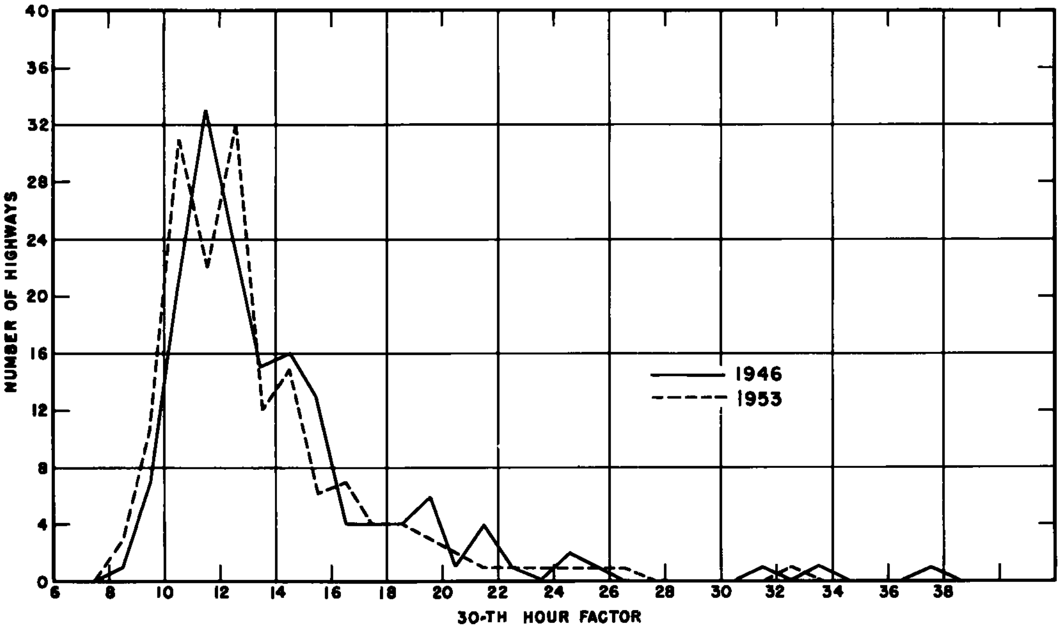

Figure 1. Distribution of 30th-hour factors for 160 rural highways of all classes in 29 states.

this small, the lowest one being 8.2 (in 1946) on a heavily traveled multi-lane facility, in a densely populated section of the country. Only 6 stations had factors in 1946 of less than 10.0. For this group of stations the change in the average factor during the 7-yr period has been almost imperceptible. This is illustrated by the lower curve in Figure 4, which shows the trend in the factor for five ranges in the values for the 30th-hour factor.

It is only for the groups of factors having values of 15.0 or more that any appreciable change in the average factor with passage of time is to be noted. The stations having the highest factors are those showing the greatest change. Average factors for stations having a 1946 factor of 30 or above have declined from 33.7 in 1946 to 27.4 in 1953, a reduction of 6.3. For stations having 1946 factors of between 20 and 30 the change in the average factor during the 7-yr period amounts to 3.0, while the corresponding change for stations having 1946 factors within the range 15.0-19.9 is 1.8. For factors of less than 15.0 (72 percent of the total), the change in the factor with passage of time is very small. This further accounts for the apparent stability of the average factor for roads having volumes of between 2,000 and 3,000 ADT (Figure 3) because of 14 of the 18 stations within this volume group had factors of less than 15.0.

The average of the factors for the 6 stations having 30th-hour factors of less than 10.0 suggests that as a practical matter the irreducible minimum value must be in the neighborhood of 9.5. There are, of course, exceptions to this rule but they are few in number and limited perhaps to facilities that are so heavily overladen that travel habits are influenced to an excessive extent by a high degree of congestion.

Average Rate of Change Affected by an Increase in Factor

It has been shown that if the stations are classified into groups according to either traffic volume or magnitude of factor, the average of the factors for any group shows at least a slight tendency toward decreasing with passage of time. It has also been stated that when the stations are considered individually, almost 30 percent of them experienced an increase in the 30th-hour factor. When the stations are grouped according to magnitude of factor some of the groups will include stations for which there was an increase in the factor, but it is evident that the effect of these stations is more than offset by the effect of the stations within the group that experienced a decrease in the factor. The groups of stations that showed the least change in the factor (factors of less than 15) were those that included most of the stations for which there was an increase in the factor. Figure 4 shows that the groups of stations having factors of less than 15 experienced very little change in the average factor. Of the 115 stations for which the factor in 1946 was less than 15.0, 41 stations experienced an increase and 74 experienced a decrease in the factor. Of the 45 stations having factors of 15 or more, only 5 experienced an increase in the factor. Thus, if the factor for any station is less than 15, there is better than one chance in three that the factor will increase rather than decrease with the passage of time. Also, if the factor is greater than 15, there is very little likelihood that it will increase at all.

A somewhat different and more refined interpretation can be made of Figure 3 after examination of Figure 4. It is apparent that the magnitude of the factor and the traffic volume act in combination to influence the trend in the factor with passage of time. Factors for roads carrying in excess of 3,000 ADT decline at a more rapid rate than those with lesser volumes, and factors of 15 or above will decline at a more rapid rate than smaller factors. The magnitude of the factor seems to exert a greater influence on the rate of change than does traffic volume. Roads with volumes in excess of 3,000 ADT in combination with factors of 15.0 or above experience the highest rate of decline in the factor. The lowest rate of decline in the factor is experienced by roads having factors of less than 15.0, regardless of the traffic volume. Perhaps there are numerous vari-

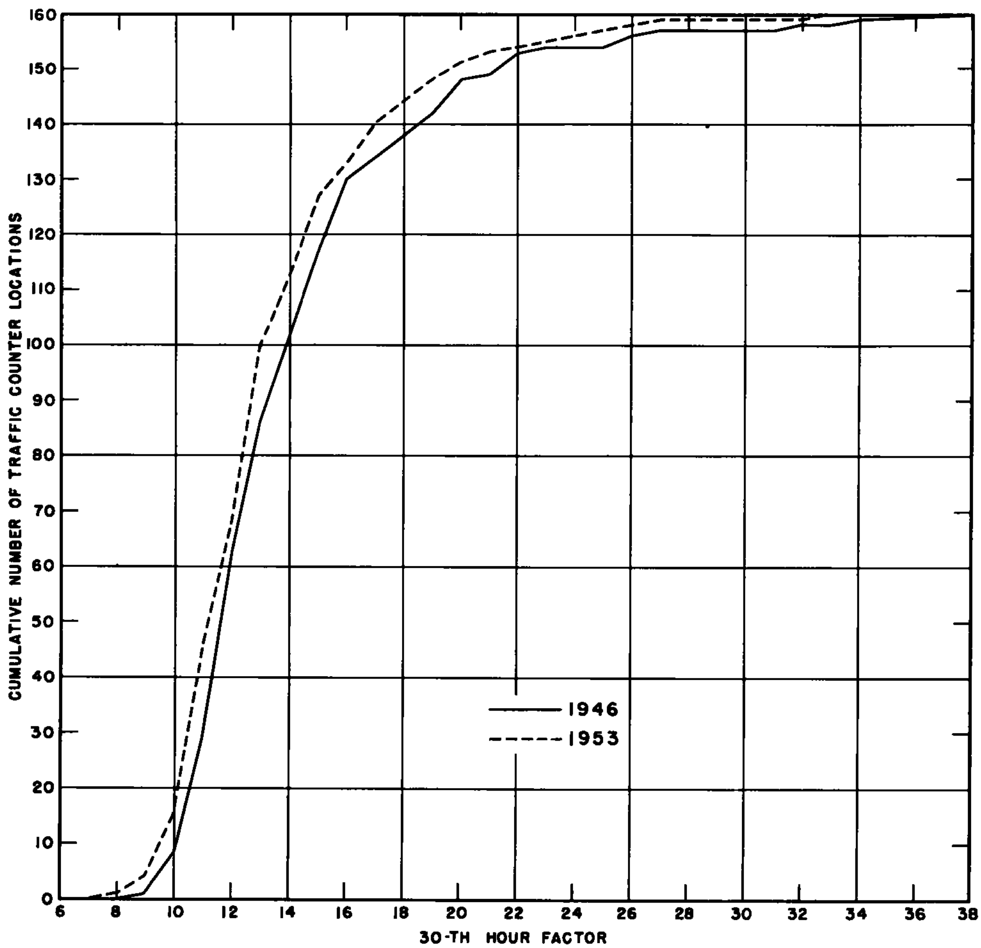

Figure 2. Cumulative number of traffic counter locations having 30-hour factors of various values.

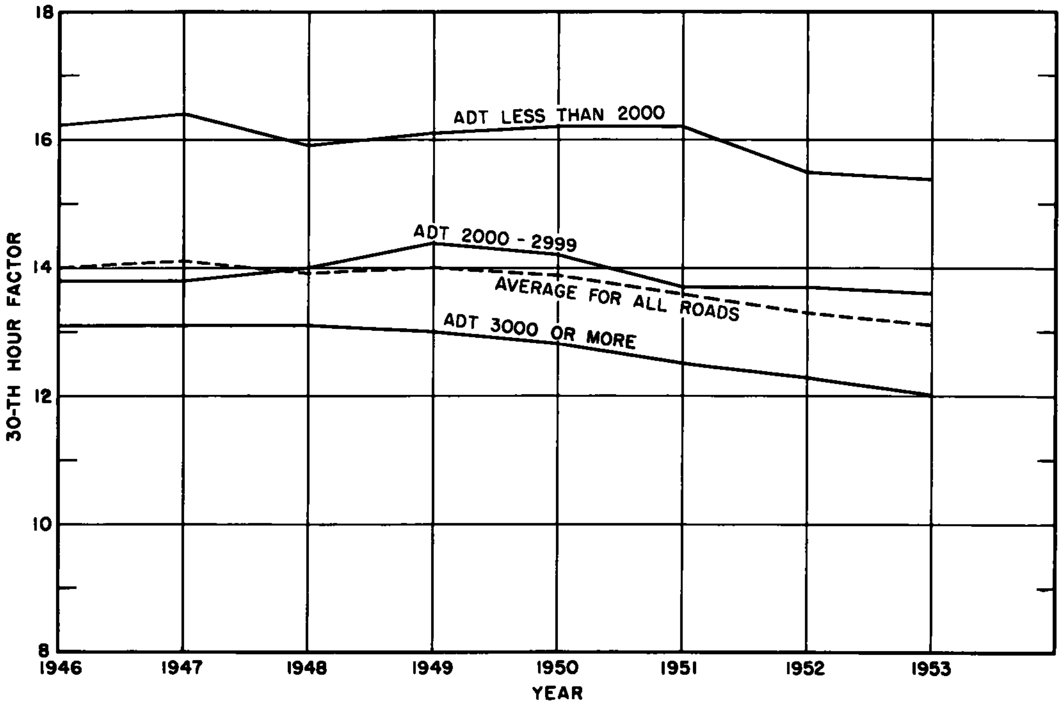

Figure 3. Trends in the magnitude of the 30th-hour factor for roads carrying various volumes of traffic.

ables other than traffic volume and magnitude of factor which influence the trend, but the amount of information available for this analysis imposes stringent limitations on their detection and evaluation.

With the passing of years, the number of stations having high factors will diminish and the average rate of decline will be reduced. When the factor for a particular faci-

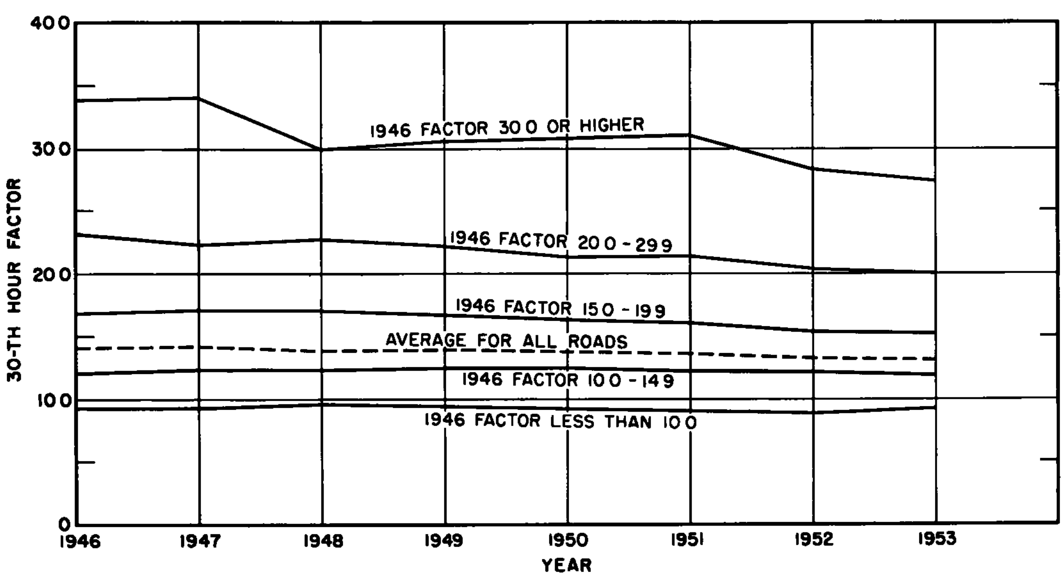

Figure 4. Trend in the 30th-hour factor for roads having factors of various magnitudes.

lity reaches a value somewhere between 9.5 and 15.0, depending upon such things as the character of traffic and geographic location, the decline in the factor will be arrested and very little change may be expected thereafter. If the conditions surrounding the minimum values lying within the range of 9.5 to 15.0 could be ascertained, the results of this analysis would be more valuable, but the data at hand does not make this possible.

TABLE 1

ANNUAL CHANGE IN 30TH-HOUR FACTOR

30th-Hour Factor	Annual Change in 30th-Hour Factor, ADT 1,200*	2,500*	6,500*
Pct. of ADT	Pct. of ADT	Pct. of ADT	Pct. of ADT
Below 10.0	0.00	0.00	-0.08
10.0 - 10.9	0.00	0.00	-0.10
11.0 - 11.9	0.00	0.00	-0.12
12.0 - 12.9	0.00	0.00	-0.15
13.0 - 13.9	0.00	-0.01	-0.18
14.0 - 14.9	-0.01	-0.02	-0.22
15.0 - 15.9	-0.02	-0.04	-0.27
16.0 - 16.9	-0.03	-0.07	-0.31
17.0 - 17.9	-0.05	-0.10	-0.36
18.0 - 18.9	-0.08	-0.13	-0.41
19.0 - 19.9	-0.10	-0.17	-0.48
20.0 - 20.9	-0.14	-0.20	-0.53
21.0 - 21.9	-0.18	-0.24	-0.59
22.0 - 22.9	-0.21	-0.29	-0.65
23.0 - 23.9	-0.25	-0.34	-0.71
24.0 - 24.9	-0.30	-0.39	-0.79
25.0 - 25.9	-0.35	-0.44	-0.83
26.0 - 26.9	-0.40	-0.50	-0.90
27.0 - 27.9	-0.46	-0.55	
28.0 - 28.9	-0.52	-0.61	
29.0 - 29.9	-0.58	-0.67	
30.0 - 30.9	-0.63	-0.74	
31.0 - 31.9	-0.70	-0.81	
32.0 - 32.9	-0.78	-0.90	
33.0 - 33.9	-0.83		
34.0 - 34.9	-0.90		

* Volumes shown in column headings are average volumes for groups of stations exhibiting annual changes in 30th-hour factor as shown in the respective columns. For volumes other than these average values interpolation between columns is recommended, as follows:

(a) For volumes of less than 1,500 ADT, use values shown in column headed "1,200";

(b) For volumes in the range of 1,500 to 2,500, interpolate between the values shown in the columns headed "1,200" and "2,500";

(c) For volumes in the range of 2,500 to 3,500, interpolate between the values in the columns headed "2,500" and "6,500";

(d) For volumes of 3,500 or over, use values in column headed "6,500."

APPLICABILITY

As in the case of many investigations of this type the best that can be hoped for is the development of broad indications which, if applied, will provide results that are more nearly correct than would have been possible of accomplishment in the absence of the study. To be of maximum value to the user, the findings of the investigation must be accurate and in considerable detail. When measured against this standard this study is somewhat deficient because it does not fully answer the question as to why traffic patterns for some roads behave differently from others in the course of time. If applied wisely, however, such facts as have been brought to light should permit a far more accurate estimate of future 30th-hour factors than would otherwise be possible.

As an aid in applying the results of the study, a table of suggested annual changes in the 30th-hour factor for various combinations of factors and traffic volumes has been prepared. These suggested annual changes (Table 1) approximate the average changes found in the statistical analysis.

EXAMPLES OF APPLICATION OF TABLE 1

Example 1

The 1956 volume on a rural road is 5,300 ADT. The 30th highest hourly volume in 1956 was 778 vph, yielding a 30th-hour factor of 14.30. It is estimated that the ADT in 1970 will be 10,600, or double the present volume. Estimated volumes for the intervening years between 1956 and 1970 are as shown in column 2 of the accompanying table. What will be the 30th-hour factor for 1970?

Year	ADT	30th-Hour Factor	Annual Change
1956	5,300	14.30	-0.22
1957	5,560	14.08	-0.22
1958	5,850	13.86	-0.18
1959	6,150	13.68	-0.18
1960	6,460	13.50	-0.18
1961	6,790	13.32	-0.18
1962	7,130	13.14	-0.18
1963	7,500	12.96	-0.15
1964	7,880	12.81	-0.15
1965	8,280	12.66	-0.15
1966	8,700	12.51	-0.15
1967	9,140	12.36	-0.15
1968	9,600	12.21	-0.15
1969	10,100	12.06	-0.15
1970	10,600	11.91	

Solution. Since the volume for all years is above 3,500, note (d) for Table 1 applies to this example. No interpolation is required. The annual change for the year 1956 is found in column 4 of Table 1. The change, which is -0.22, is applied to the 1956 factor of 14.30 to yield a 1957 factor of 14.08. Thirtieth-hour factors for succeeding years are obtained in a similar manner. The rate of change diminishes as the factor becomes smaller.

Example 2

The 1956 volume on a rural road is 1,400 ADT. The 30th-hour volume in 1956 was 259 vph, yielding a 30th-hour factor of 18.50. The estimated rate of traffic growth is about 10 percent per year for each of the next 14 years (column 2 in the accompanying table). What will be the 30th-hour factor for the year 1970?

Year	ADT	30th-Hour Factor	Annual Change
1956	1,400	18.50	-0.08
1957	1,550	18.42	-0.08
1958	1,700	18.34	-0.09
1959	1,875	18.25	-0.10
1960	2,050	18.15	-0.11
1961	2,250	18.04	-0.12
1962	2,500	17.92	-0.10
1963	2,750	17.82	-0.17
1964	3,000	17.65	-0.23
1965	3,300	17.42	-0.31
1966	3,600	17.11	-0.36
1967	3,950	16.75	-0.31
1968	4,400	16.44	-0.31
1969	4,800	16.13	-0.31
1970	5,300	15.82	

Solution. Since the ADT changes from an ititial value of less than 1,500 to a final value of over 3,500, notes (a), (b), (c) and (d) for Table 1 apply in this example. The factor for the year 1957 is obtained by subtracting the annual change for 1956 (0.08, column 2, Table 1) from the 30th-hour factor for 1956 (18.50). The annual change is here obtained in accordance with note (a). The procedure is repeated for each succeeding year to 1970, with the resulting 30th-hour factor for 1970 being 15.82. During the years 1957 through 1961 the ADT is between 1,500 and 2,500 and the annual change is obtained by interpolating between 0.08 in column 2 of Table 1 and 0.13 in column 3, in accordance with note (b). During the years 1963 through 1965 the ADT is between 2,500 and 3,500 and the annual change is determined by interpolating between 0.10 in column 3 and 0.36 in column 4, in accordance with note (c). For the years 1966 through 1969 the volume is greater than 3,500 and no interpolation is required. For these years the annual change is taken directly from column 4 of Table 1 in accordance with note (d).

SUMMARY

1. The trend in the 30th-hour factor for 160 stations in continuous operation over a 7-yr period has been a decline of 0.11 per year, on the average, but the rate of decline varied widely between different roads.
2. Roads having volumes of more than 3,000 ADT in combination with factors of 15 or above are those which, as a class, experience the most rapid rate of decline in the factor.
3. The lowest average rate of decline (almost zero) is experienced by roads having 30th-hour factors of less than 15.0, regardless of traffic volume.
4. The fact that there is little change in the average factors for the groups of roads having factors of less than 15.0 is due in part to the phenomenon that within these categories there is almost a one-in-three chance that any given station will experience an increase rather than a decrease in the factor with passage of time.
5. The irreducible minimum value for the 30th-hour factor may be about 9.5 for rural highways. In some geographical areas, and under certain conditions yet to be defined, the minimum value of the factor may be as high as 15.0, but a factor lower than 9.5 may be accepted as a definite indication that travel desires are being supressed.

REFERENCES

1. Peabody, L. E., and Normann, O. K., "Application of Automatic Traffic Recorder Data in Highway Planning," Public Roads, Vol. 21, No. 11 (January, 1941).
2. "Highway Capacity Manual," Part VIII, U. S. Dept of Commerce, 1950.

Appendix

STATIONS USED IN ANALYSIS OF TRENDS IN 30TH-HOUR FACTOR

State	No. of Stations	Identification of Stations
Conn.	18	1,3,4,5,6,8,9,10,11,13,15,16,18,20,21,22,25,27.
Del.	6	1A,2B,3C (N+S),4D, 5E,7G.
Ga.	4	1,3,4,12.
Idaho	2	3,7.
Ind.	6	34A,42A,47A,59A,72A,73A.
Ill.	3	2,7,8.
Iowa	4	601,604,614,616.
Maine	5	2,3,4,7,9.
Md.	16	1,2,3,4,5,6,7,8,9,10,11,12,13,14,15,18.
Mass	5	1,2,10,12,13.
Miss	4	3,8,10,4A.
Mich.	5	600,603,606,607,617.
Mo.	6	1,2,5,12,22,26.
Neb.	5	A2,A4,A5,A8,A12.
Nev.	4	101,107,109,110.
N. H.	5	1,2,4,6,15A
N. C.	6	1,2,3,4,22,24.
Ohio	4	25,28,31,38.
Okla.	16	1,4,5,6,8,9,10A,11,12,13,14,15,17,18,19,20.
Pa.	7	4,5,10,17,21,22,31.
Tenn.	2	3,11.
Texas	5	1,4,5,8,17.
Utah	7	301,302,305,308,312,313,315.
Vt.	3	A12-1,C14-2, PR110.
Wash	9	1,2,3,6,7,8,12,14,15.
W. Va.	3	1,3,4.
Total	160	

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