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CONTENTS

FOREWORD

Discussions of ways to evaluate qualities of traffic service, as well as studies of capacity and level of service for two-lane highways, junctions, and weaving areas, make up the three papers and three abridgments in this RECORD. The findings should be useful to highway engineers and others involved in estimating or measuring levels of service so as to provide maximum utility of highways.

The first abridgment briefly describes Claffey's work to determine the practicality of using precision fuel-consumption measurements to rate the quality of service of urban streets. Although it was concluded that the system could be an effective and convenient study tool, further research is indicated to define its specific utility and applicability.

Courage and Bissell report on the development of a traffic survey system that collects data on a digital tape system and provides off-line computer analysis. Recording and analysis of traffic engineering measures is described with two examples, one stationary that uses road tubes at the roadside, and one in a moving vehicle. Speed, flow, and density contours, delay contours, speed and delay peak-hour profiles, travel time plots, volume, and headway are among the data developed in the example studies, and other applications are reported to be under development.

A more efficient method of studying intersection operation was the aim of the study by Diewald and Nemeth, reported in the next abridgment. Field studies were made with cameras and mirrors at four types of intersections to provide validation data for a computer simulation model capable of simulating the intersection of two- and four-lane roadways with and without left-turn lanes. The authors observe that the model was validated on the basis of delay information from the photographic data system.

Pignataro and others report on their extensive study of weaving-area operations as related to the weaving section procedures of the 1965 Highway Capacity Manual. Analysis of previously acquired data led to results indicating that the predicted level of service quite frequently differs from the actual service level. They recommend a reconstitution of the weaving procedure to be applied to both the major-weave and auxiliary lane ramp-weave cases and specify a data program to allow calibration of the recommended procedure. A discussion from an experienced practitioner supports the conclusions drawn by the researchers.

The abridgment of the work by Wattleworth and Ingram describes a linear programming model for determining the capacity of freeway interchanges as a function of geometric configuration, physical characteristics, and traffic patterns. Designers should find the model useful in determining capacities of alternate interchange configurations, as well as in the analysis of the capacity of signalized intersections.

The final paper by Rørbech reports studies on four rural two-lane highways in Denmark to determine whether provisions for two-lane highways given in the Highway Capacity Manual were valid there. The principal difference detected was the higher operating speeds in Denmark, and their data allowed construction of curves to be used in planning roads in the near future.

PRECISION FUEL-CONSUMPTION MEASUREMENT FOR RATING THE QUALITY OF SERVICE OF URBAN ROUTES

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ABRIDGMENT

•A PILOT study to determine the practicality of using precision fuel-consumption measurements to rate the quality of service of urban streets was carried out in downtown Potsdam, New York, during 1971. The use of precision fuel-consumption measurements to rate traffic service levels is based on the concept that, in general, each traffic factor responsible for travel inconvenience, delay, discomfort or combinations of these also causes a minute increase in fuel consumption. It is possible with precision fuel-consumption measurements to detect the accumulated excess fuel consumption attributed to these factors. These measurements could provide a convenient means for rating the quality of service of urban routes.

In the pilot study, a passenger car equipped with a precision electronic fuelmeter was operated at least 10 times over each of three downtown routes between common end points. Each route had a different set of conditions affecting traffic flow. Fuel consumption to the nearest 0.001 gal and time to the nearest 0.1 sec were recorded for each trip as well as a count of major speed changes and of evasive traffic maneuvers executed. The precision electronic fuelmeter used in this study is described elsewhere (1, p. 83).

The pilot study provided definite evidence that precision fuel-consumption measurement could be used as an effective and convenient means of rating quality of traffic service. There is a close relationship between quality of service ratings found by this measurement and quality of service as evidenced by regularity and smoothness of travel. Precision fuel-consumption measurements are sensitive not only to large speed changes, prolonged operation at reduced running speeds, and stopped time but also to the multitude of small changes in direction and speed that continually plague vehicles in traffic. Furthermore, for an attempted speed of 30 mph (representative of urban routes), each traffic factor affecting vehicle operation to the detriment of travel comfort and convenience also tends to increase fuel consumption.

However, a much broader research study is required to define the specific utility and applicability of this investigative technique for developing information on the service quality of urban routes. Further test operations should include the use of equipment for detecting and recording small speed changes (on the order of 2 to 3 mph). The frequency of small speed changes and their value ranges can be combined with the frequency of stop-and-go and other large speed changes for evaluation of the sensitivity of fuel-consumption measurements to speed changes.

The development of means for precision fuel-consumption measurement has made possible a new method for rating the quality of service of highways, particularly urban routes. Precision fuel-consumption measurement is attractive for rating quality of service because the needed measurements not only are easy to make but also reflect directly the accumulated effect of most traffic factors that affect travel comfort.

REFERENCE

1. Claffey, P. J. Running Costs of Motor Vehicles as Affected by Road Design and Traffic. NCHRP Rept. 111, 1971.

RECORDING AND ANALYSIS OF TRAFFIC ENGINEERING MEASURES

Kenneth G. Courage*, University of Florida; and
Howard H. Bissell*, Federal Highway Administration

This paper deals with the development of a traffic survey system that collects data on a digital tape system and provides off-line analysis by putting the tape on a general-purpose electronic digital computer. Examples of two methods that have actually been conducted in the field are presented. One example involves the use of the recorder in a moving vehicle to collect data on time, distance, and approaching vehicles passed on a freeway facility. These data are used to develop speed, flow, and density contours; delay contours; speed and delay peak-hour profiles; isochronal travel time plots; and bottleneck flow density relationships. The second example involves stationing the recorder at the side of the roadway and collecting speed, volume, and headway data through the use of road-tube detectors. These data are used to compute speeding, tailgating, and turbulence indexes. Other traffic engineering study applications are being developed.

•THE DESIGN of traffic control systems is highly dependent on traffic operations data. Many of the techniques used for collecting and analyzing these data do not fully satisfy the information requirements of today's traffic control systems, nor do they take full advantage of the automation now available for collecting and processing this information. These deficiencies have seriously retarded the development of the more advanced traffic control systems required to solve our increasingly complex traffic problems. In other words, some of the traffic congestion on our streets today could be eliminated by development of better data collection and analysis techniques.

Many studies made by manual observers or simple recording devices deal mainly with system inputs and outputs and do not provide for an assessment of the interaction between the vehicle and the control system or among several vehicles within a given system. More comprehensive studies, using aerial photography, for example, have been used successfully to provide detailed information; however, their application has been somewhat limited because of factors such as cost, complexity, and manual data reduction requirements. Another possible tool for this purpose is the digital magnetic tape recorder, which may be used to record data from several sources, e.g., detectors, odometers, signal controllers, and manual switches. These data may be analyzed directly on a digital computer without the time-consuming manual reduction steps. This technique is suited to a wide variety of traffic operations analyses.

This paper deals with two specific applications of a prototype magnetic tape recorder system developed for recording and analysis of traffic engineering measures (RATEM). In the first example, the recorder is carried within the vehicle and is used to record "floating-car" data. In the second example, the recorder is left in a stationary position and connected to temporary road-tube sensors to collect traffic flow data.

SYSTEM OPERATION

The data recorder was designed around a commercially available incremental tape unit. This unit operates on 12 Vdc, uses ½-in. computer tape, and writes 6 channels plus parity at 200 bytes per inch. It is provided with internal logic for writing the

*At the time this research was performed, the authors were with Kelly Scientific Corporation.

contents of its input register (6 bits) on the tape upon command. For analysis purposes, these 6-bit words are grouped into "records," each containing 2,048 words. The traffic inputs are required to be in switch closure form. Special interface circuitry was constructed in the prototype version to accommodate inputs such as road tubes, presence detectors, odometer pulses, traffic signal circuits (115 Vac), and manual event switches.

The prototype was designed to record 18 bits of information each time any one of the inputs was actuated. These 18 bits appear on the tape as three 6-bit words. One word was used to indicate which input switch was activated. Each traffic input circuit was assigned a unique 6-bit code, giving the system the capacity to accept 64 independent inputs. In the prototype unit, however, the necessary hardware was provided for only 16 such inputs.

The remaining 12 bits (2 words) are used to indicate the time at which the actuation took place. The time value is advanced by one unit each 2 millisec by an internal oscillator. Thus, the clock recycles approximately every 8 sec ($2^{12} \times 0.002$). The recycling of the clock is treated in the same manner as an input switch closure; i.e., an 18-bit word is recorded with the 6-bit device code identifying the clock. No record is kept of the actual time of day; however, this value may be computed by the analysis program by counting the clock recycling periods, provided that the initial time of day is known. Figure 1 shows that the 18-bit word is first entered into a memory buffer with a six-word capacity and finally recorded on the tape by the internal logic of the tape drive. The memory buffer prevents the loss of data when the input data rate temporarily exceeds the recording speed of the tape unit. This configuration promotes the most economical use of tape inasmuch as each actuation writes only three words, and no data are written except upon actuation. This permits unattended operation for several days at locations where detector impulses are being recorded.

MOVING-VEHICLE STUDIES

Detailed studies have been carried out to determine the patterns of speed, flow, density, and delay for traffic on a freeway or urban surface street through use of RATEM system hardware (1). The system was installed in a vehicle, and traffic measurements were made as the vehicle traveled through traffic. Distance measurements were recorded from "fifth-wheel" pulses directly onto the magnetic tape.

Study runs were made alternately with the system-equipped vehicle traveling "with" and "against" the direction of traffic movement being measured. When traveling with the traffic, the computer recorded each stopped vehicle that was passed (e.g., on a freeway with exit-ramp queues) as an event on the magnetic tape. When traveling in the opposite direction, the computer recorded as events all vehicles in the major traffic stream as they were passed. The data obtained in this manner for a particular roadway section of length (L) include the speed of the traffic movement of interest, V_f; the speed of traffic in the reverse direction, V_r; the number of vehicles counted within the section, N_o; and the number of stopped vehicles queued with the section, N_q.

One of the variables of interest, speed, was obtained directly from these data as V_f. The other two, flow Q and density K, may be calculated as follows:

$$K = N_q + (N_o - N_q)\,[V_r/(V_r + V_f)]/L$$

$$Q = KV_f$$

These equations may be applied only on two-way routes where the view of the oncoming traffic is not obstructed.

These calculations were performed automatically as the data tapes were read by the computer. Survey section speed, flow, and density data were then analyzed to produce the following results:

1. Speed, flow, and density contour plots;
2. Peak-hour profiles of speed and density;

3. Contour plots of delay to the freeway traffic;
4. Peak-hour travel relationships;
5. Isochronal travel time plots; and
6. Bottleneck flow-density relationships.

As an example of the moving-vehicle technique, a portion of the results of studies conducted on the Jones Falls Expressway in Baltimore, Maryland, is shown in Figures 2 through 6.

Speed, Flow, and Density Contours

Speed, flow, and density contours for the north section of the Jones Falls Expressway for southbound a. m. peak hours are shown in Figure 2. Minor congestion is evident in this figure for a short time near the downstream limit of the study. The main reason for this congestion is the introduction of two entrance ramps immediately upstream. The speeds and densities indicate that near-capacity operation is experienced at this point and that the upstream section of the freeway is quite capable of handling the traffic demand under stable flow conditions.

Speed and Density Profiles

The operation of this facility is described in the macroscopic sense by the speed, flow, and density contours derived from the data. It is somewhat difficult, however, to carry the interpretation of these contours much beyond a narrative description. Some further analyses may be performed on these data. One meaningful way of quantifying the operation is a profile plot of the average speed and density over an entire peak period. For example, Figure 3 shows speed and density profiles for the heaviest hour of the a. m. peak period (7:45 to 8:45). These profiles are useful in assessing the approximate degree of congestion throughout the length of the freeway and in identifying bottleneck characteristics. The mirror-image characteristics are particularly interesting inasmuch as the peaks of wide separation between the speed and density curves indicate the existence of bottlenecks. The most pronounced peak is observed at the southern exit ramps, where considerable congestion is known to occur. Similar, but lesser, bottlenecks are noted in the areas of the North Avenue and the 28th Street exit ramps. Another minor bottleneck occurs around mile 3.7. This may be due to the roadway curvature in this area or to the sun glare that is sometimes present during the morning peak. Some congestion is also observed close to the Cold Spring interchange where another divergence between the speed and density curves is noted.

Delay Analysis

Delay is another useful measure that may be derived from the speed, flow, and density data. There are several definitions of delay; the definition best suited for this study is as follows: "Delay, measured in vehicle-minutes, is the excess travel time expended by all vehicles on the freeway because of an operating speed lower than the desired value." Setting the desired speed at 40 mph, which is close to optimal for maximum capacity, permits the calculation of delay D for a given section of roadway as follows:

$$D = 60\ (L/V - L/40)\ \text{min/mile/vehicle}$$

where

L = length of the section, and
V = speed of traffic.

Figure 4 shows the delay contours for the morning peak period on the Jones Falls Expressway. The delay reaches a peak value of about 3 min/mile at about 8:15 a. m. and concentrates in the area of the south exit ramps.

Figure 1. Simplified data flow diagram for the RATEM system.

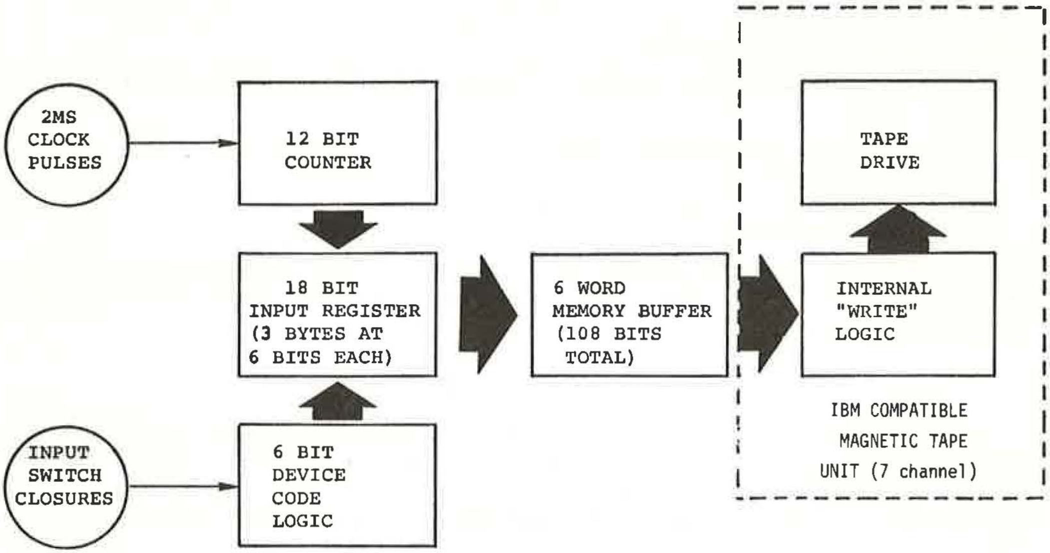
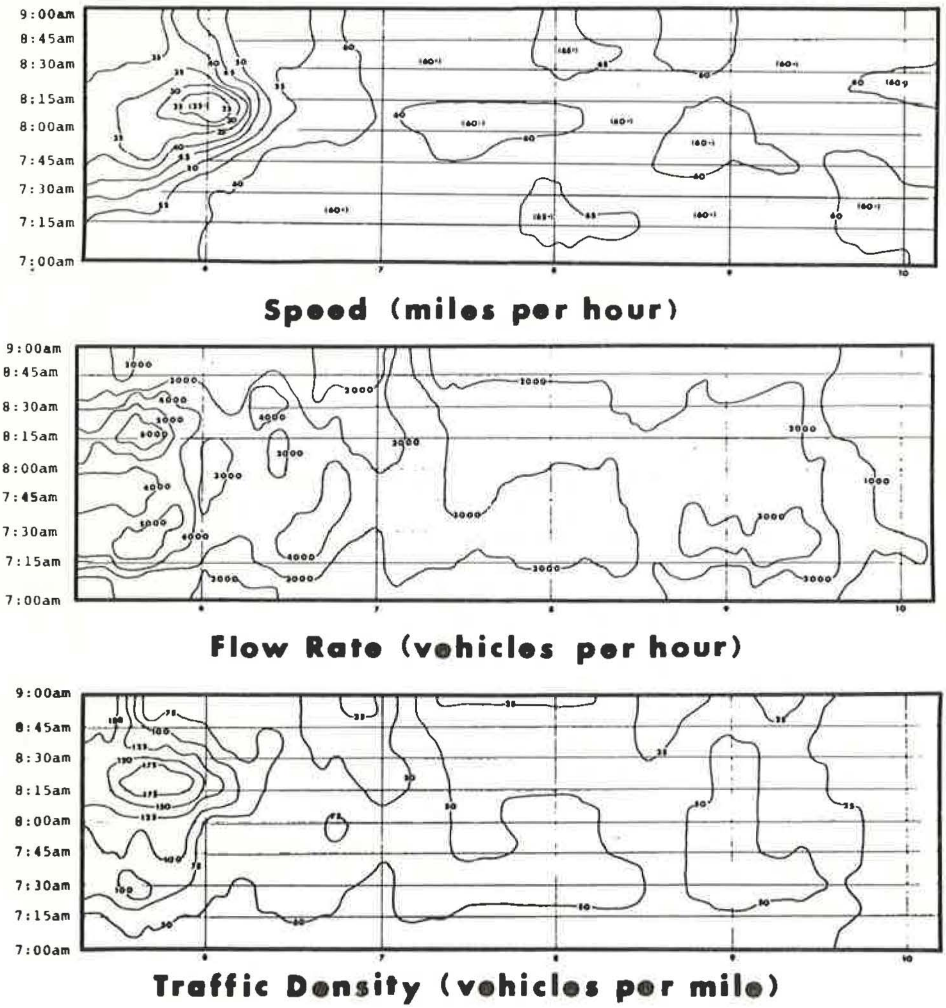


Figure 2. Speed, flow, and density contours, a.m. peak period.



Peak-Hour Travel Relationships

Four factors of interest are readily obtainable from the peak-hour travel relationships:

1. Total travel (vehicle-miles), TT;
2. Total travel time (vehicle-hours), TTT;
3. Total delay (vehicle-hours), D; and
4. Average speed (mph), V.

Total travel TT was calculated by summation of the flow measured on each section of the freeway over the 1-hour period, multiplied by the length of the section, which in this study was $\frac{1}{20}$ mile; i.e.,

$$TT = \frac{1}{20} \Sigma Q$$

Total travel time TTT was calculated by integrating the values of traffic density over both time and distance. This figure may be represented, therefore, by the total volume of space contained under the density contours.

Total delay D was calculated by the summation of the individual values

$$D = TTT - TT/40 \text{ mph}$$

obtained for each 5-min sample on each $\frac{1}{20}$-mile section within the study limits.

Average speed V was calculated by simply dividing the total travel by the total travel time; i.e.,

$$V = TT/TTT$$

The values described as determined for the southbound a. m. peak period for the Jones Falls Expressway are as follows:

Factor	Value
TT, vehicle-miles	17,764
TTT, vehicle-hours	484
D, vehicle-hours	66.6
V, mph	36.7

Isochronal Travel Time Plots

Another measure of interest in assessing roadway operations is the expected travel time for each vehicle from any point on the highway to a reference point, usually the end, as a function of the time of day as well as the distance from the reference point. These measurements are shown in Figure 5 for the a. m. peak period for the southbound Jones Falls Expressway. It is noted that the time to travel the study freeway link varies from 5 to 7 min during the a. m. peak.

Bottleneck Flow-Density Relationships

The relationship between flow and density on a section of highway is useful in determining the optimal traffic density that should be allowed on the facility as well as the capacity, or peak flow, that can be accommodated. The flow-density curve for the bottleneck area of the freeway in question is shown in Figure 6. The curve uses 5-min samples over $\frac{1}{20}$-mile sections. Points are plotted individually for each 10-vehicle/mile density increment. The maximum flow rate of 5,400 vph (3 lanes) was obtained at densities of about 170 vehicles/mile or about 57 vehicles/mile/lane. These figures fall within the range of generally accepted values.

Figure 3. Speed and density profiles, a.m. peak period.

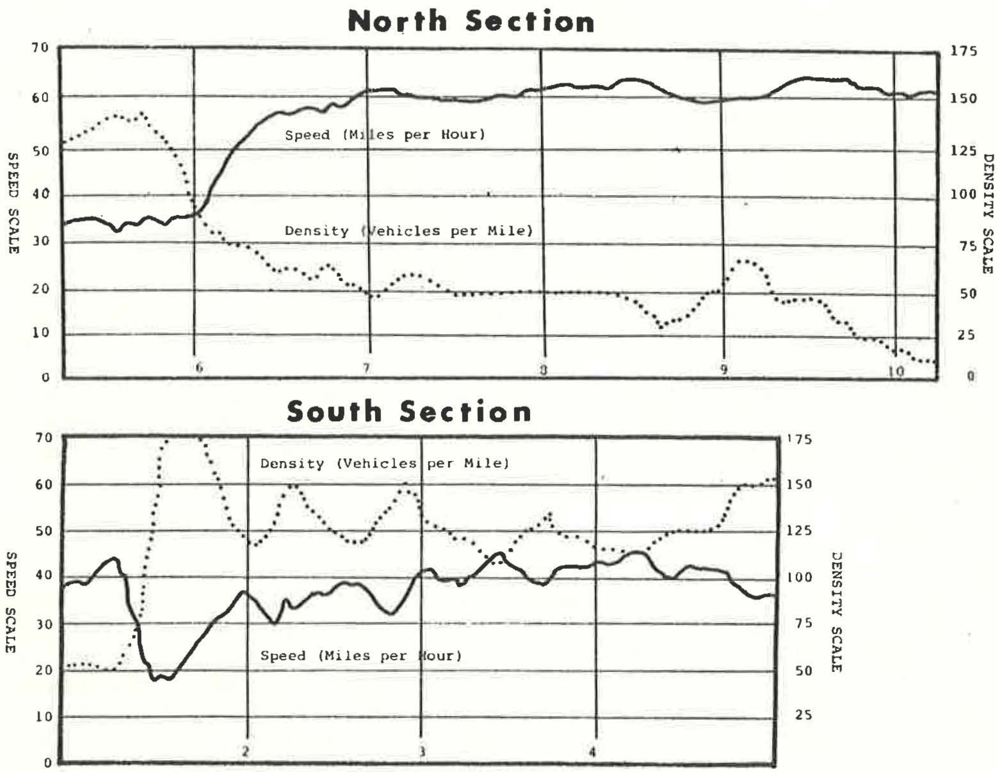


Figure 4. Delay contours, a.m. peak period.

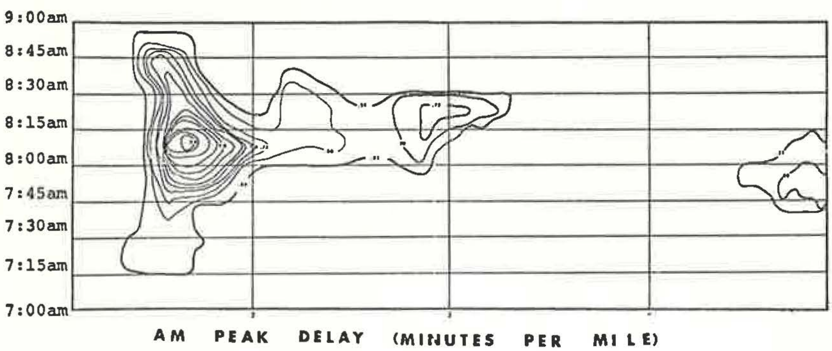
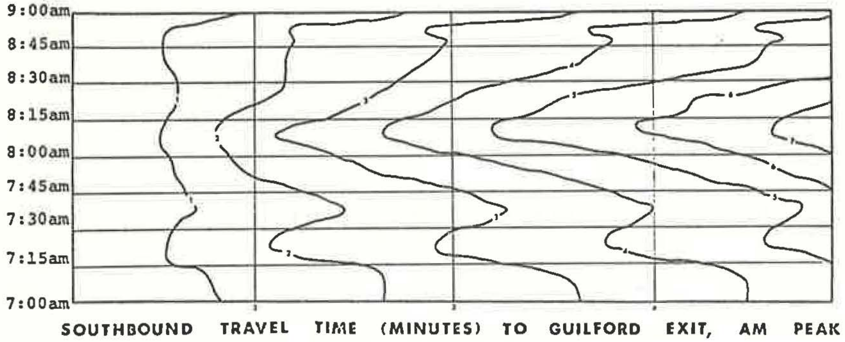


Figure 5. Travel time plots, a.m. peak period.



STATIONARY STUDIES

Data Collection

The RATEM system can be set up beside a roadway to collect a variety of traffic data by using road-tube or tapeswitch inputs. This application was initially developed for the National Highway Safety Bureau as a possible approach for sampling and recording traffic violations (2). The digital recorder was connected to temporary road-tube sensors to detect and record a variety of traffic violations and safety information. The types of data collected depended primarily on the configuration of the detectors.

The detector configurations used in the NHSB studies are shown in Figures 7 and 8 for a two-lane, two-way roadway and a four-lane, two-way roadway. The tubes were placed 4 ft apart, and the time actuations of a vehicle crossing each tube were recorded by the magnetic tape unit.

Because only actual events, axle crossings and time marks, were recorded on the magnetic tape, a single tape could record data for a complete 3-day (24 hours per day) period at a location that handled about 30,000 vehicles/day. The equipment could be left unattended; however, it was found that periodic checks were desirable.

Data Reduction

The raw data, as described earlier, were obtained in the form of axle-crossing times for all of the road-tube sensors. To convert these data into usable traffic information required that several operations be performed by the analysis program. The axles had to be identified with a particular lane on the roadway. In the case of roadways with a single lane in each direction, this was easily accomplished by reference to the location of the actuated detector. Where multiple-lane configurations were involved, it was necessary to use additional logic to determine the lane of operation. As an example, an actuation on detector B (Fig. 8) alone would indicate a vehicle in the inside lane, whereas near-simultaneous actuations on detectors A and B would indicate a vehicle in the outside lane. This concept may be extended to any number of lanes, provided that sufficient detectors are installed. Additional lanes complicate the logic considerably, however, and cause ambiguities that require data to be discarded, reducing the sample size somewhat. Where several lanes are involved it is generally preferable to use separate tapeswitches for each lane of traffic.

It is also necessary to associate individual axles with vehicles for most of the traffic measurements. An axle was considered to be associated with the preceding axle in the same lane if, and only if, spacing between the axles was less than a prescribed distance (approximately 25 ft), and the two axles were traveling at the same speed. Where more than two axles were associated with the same vehicle, additional checks were performed, based on combinations of axles, to estimate the composition of the multiaxle unit (e.g., one truck, two passenger cars, and so forth).

Traffic Measurements

The stationary study is particularly an ideal candidate for analyzing safety indexes. In the system developed and tested in the NHSB project, the computer program presented detailed analytical information that can be used directly by traffic engineers and that includes standard information such as the speed pace (the 10-mph increment that includes the largest percentage of motorists), the 85th percentile speed (the speed that is often used to set legal speed limits), and the percentage of motorists who were traveling faster than the current legal speed limit. In addition, three indexes that may be used as a measure of safety for the traffic using the roadway were developed. One of these, speeding index Sp, is calculated as

$$\mathrm{Sp} = \left[\frac{\Sigma[(\text{speed of violators}) - (\text{speed limit})]^2}{(\text{sample size of all vehicles})} \right]^{0.5}$$

Figure 6. Flow-density relationship.

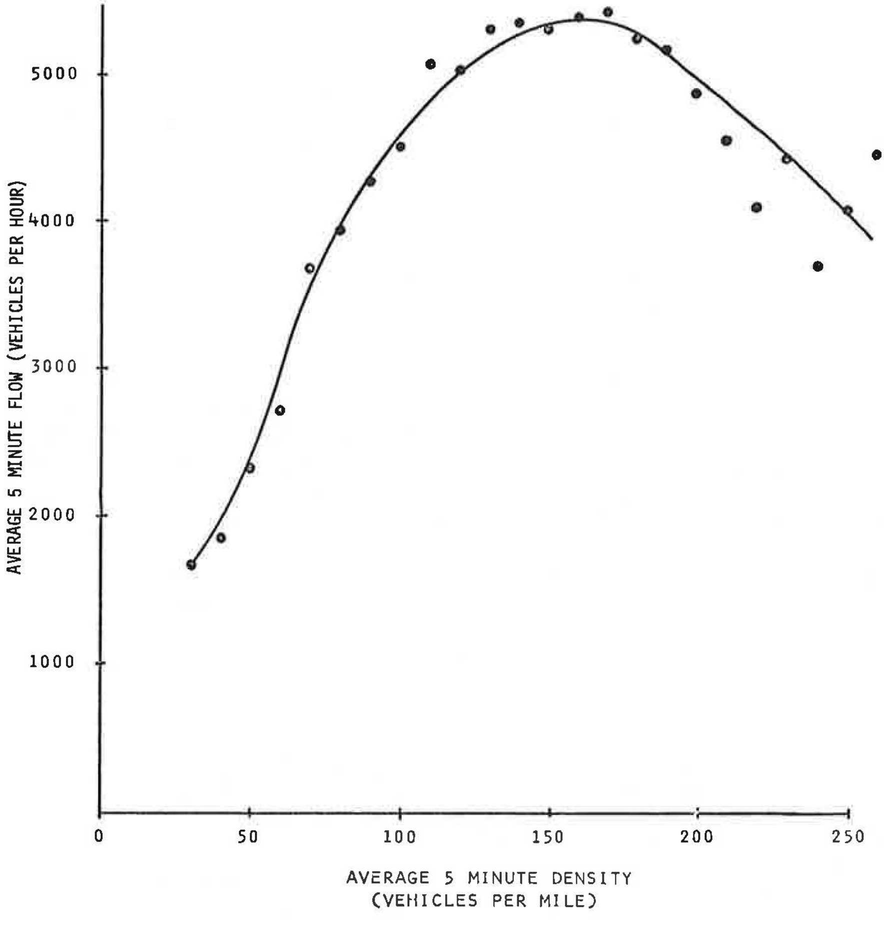


Figure 7. Multiple-tube configuration for two-lane, two-way roadway.

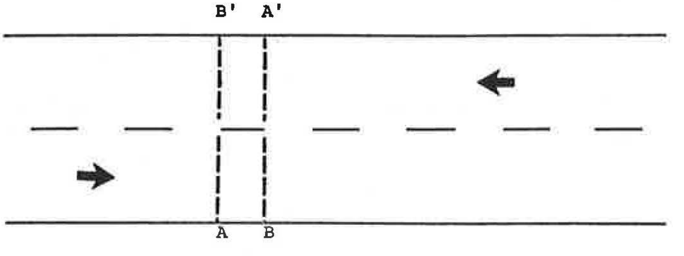
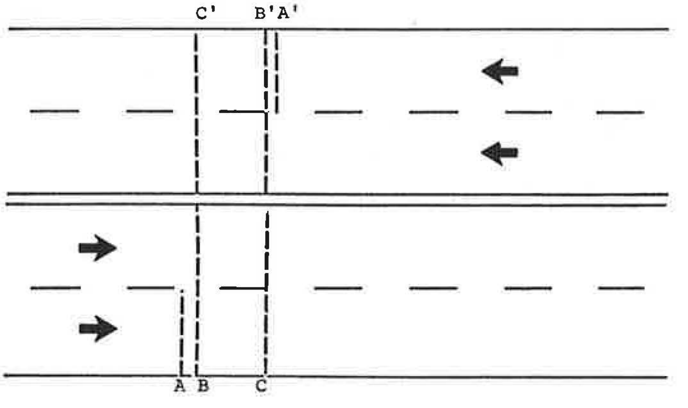


Figure 8. Multiple-tube configuration for four-lane, two-way roadway.



This index is the root mean square of the excess of speed of individual vehicles traveling faster than the speed limit. This gives greater weight to vehicles that are traveling at a speed much higher than the limit than to those who are barely speeding. The index should provide a good indicator of where speed enforcement measures should be applied. A high index figure would indicate a time and location with an extensive speed violation problem.

The tailgating index Tg is computed as

$$Tg = \left[\frac{\Sigma[(\text{allowable following distance}) - (\text{actual following distance})]^2}{(\text{sample size of all vehicles})}\right]^{0.5}$$

and is the root mean square of the deficiency in following distance of the vehicles considered to be "following too close" for safety. This may have a direct safety implication because it shows the potential for rear-end collisions. The allowable following distance was considered to be 1 ft for each foot per second of speed (roughly one car length for each 10 mph). Thus, the allowable following distance is equivalent to a time gap between vehicles of 1 sec. A high index figure would indicate an area with a potential safety problem.

The third index, turbulence index Tb, is computed as

$$Tb = \left[\frac{\Sigma(\alpha^2) - (\Sigma\alpha)^2/\text{sample size}}{\text{sample size}}\right]^{0.5}$$

where $\alpha = K\left[\frac{(\text{speed of lead vehicle}) - (\text{speed of following vehicle})}{(\text{position of lead vehicle}) - (\text{position of following vehicle})}\right]$

K is a constant, which in this study equaled unity.

The turbulence index expresses the standard deviation of acceleration α of all vehicles in the sample that passed the point. The α term was estimated by using the linear car-following model shown above. If all vehicles were traveling at or about the same speed, α would be zero or very small, and of course the index would also be small. On the other hand, if vehicles were traveling at a wide range of speeds and their respective distances were small, this would indicate that speed changes would be required to avoid collisions and the variance in acceleration would be relatively high indicating hazardous and possibly unstable flow.

The actual meaning of these values with regard to traffic safety is not yet known; however, it could be easily determined, provided that a sufficient number of studies were conducted at a variety of sites and the resulting indexes were compared to the accident records for these sites.

In the sample of the computer output, the location studied was a four-lane, two-way street. The speed limit was 50 mph. Each hourly period is analyzed, and the sample size is stated. The pace (the 10-mph speed range, which contains the majority of measured speeds) is presented with the percentage of the sample included in this speed range. The speeding violations are presented in percentage above the limit, max, and percentage below a minimum speed limit, min. Because there was no minimum at the study site the min column is 0.0. The 85th percentile speed is presented to the nearest 2-mph increment. The speeding, tailgating, and turbulence indexes were determined as described previously.

Wrong-way movements recorded if any are measured. These include the movement of a vehicle that crosses over the centerline of the roadway. Other events, such as shoulder use, can be recorded also, if needed.

SUMMARY AND CONCLUSIONS

The two examples presented in this paper illustrate the use of a digital magnetic tape system in the recording and analysis of traffic engineering measures. There are several other applications in which the same hardware configuration could be employed to collect traffic data that would be extremely useful to the traffic engineer in the planning, design, and evaluation of traffic engineering improvements. Examples of these other applications include arterial moving-vehicle studies to optimize progression, stationary studies at traffic-actuated signals to improve the efficiency of operation, and entrance-ramp merging area studies to assess the need for ramp controls.

The main purpose of the prototype system described herein was to determine the feasibility of making useful traffic operations measurements. As noted by the results presented in this paper, a large amount of valuable information can be obtained from a few moving-vehicle runs or, in the case of the stationary studies, from a relatively short field study. The time, effort, and cost for reducing the raw data are minimized by the direct use of the digital computer, which uses relatively simple analysis programs.

ACKNOWLEDGMENT

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INVESTIGATION OF A COMBINED PHOTOGRAPHIC AND COMPUTER-SIMULATION TECHNIQUE FOR USE IN THE STUDY OF ISOLATED INTERSECTIONS

Walter Diewald*, Virginia Polytechnic Institute and State University; and
Zoltan A. Nemeth, The Ohio State University

ABRIDGMENT

•THE objective of this study was to develop a new intersection study technique. The specific aims were the following:

1. To develop a photographic data-collection system that is capable of recording continuous intersection traffic data, including information on stopped time delay of left-turn vehicles and on left-turn gap-acceptance characteristics;
2. To develop an intersection simulation model so that, with information provided by the data-collection system, the intersection of two- and four-lane roadways, with or without left-turn channels, can be simulated; and
3. To carry out tests necessary to determine the validity of the simulation model.

THE PHOTOGRAPHIC TECHNIQUE

The use of photography for traffic engineering studies is desirable for a number of reasons: (a) Photographic data collection provides a continuous and permanent record of all the events taking place within the view of the camera; (b) photography helps to minimize error that can result from manual data collection; and (c) furthermore, a properly developed photographic data-collection procedure enables the researcher to collect data with a minimum of support personnel.

In past studies that made use of photographic data collection and that were concerned with intersection operation, the importance of camera placement was readily recognized, and the difficulty of obtaining satisfactory data on more than two approaches simultaneously was not overcome. As a result of this study, a unique camera and mirror system was developed that, with the use of two robot motor-recorder cameras, can photograph four intersection approaches simultaneously. This camera, manufactured in West Germany, is a compact 35-mm camera and is available with a 200-ft film magazine. Utilizing an estar-based Kodak linagraph film, the camera has a maximum capacity of recording more than 30 min of continuous data at the rate of one frame/sec. Each robot camera and mirror unit is housed within an aluminum frame that is attached to a tripod head. The tripod head is inserted into a pipe pole mount, which is then secured to a pole by two U-bolts, and a firm grip is ensured by rubber sheeting positioned between the U-bolts and the pole and also between the steel plate and the pole. The entire camera mount system for one camera (Fig. 1) provides a maximum of adjustment movements as well as portability and ease of use. Figure 2 shows a typical photograph.

Other components of the data-collection system include an intervalometer that provides the pulsing mechanism necessary for the actuation of the camera shutters every second; a 24-V power supply for the film-advance motors; a frame counter that provides a means for recording signal indication and frame count; and a specially designed control box for the camera and power actuation.

*Mr. Diewald was with the Ohio State University when this study was conducted.

Figure 1. Camera mount system.

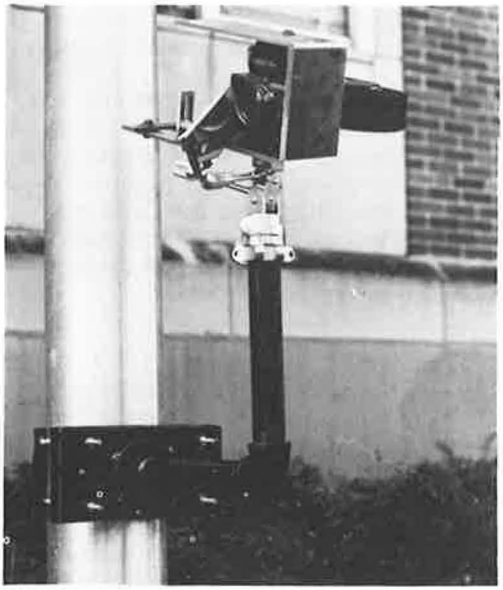

Figure 2. Sample photograph.

Data were collected at four types of intersections: single-lane approaches with and without left-turn channels and two-lane approaches with and without left-turn channels. The specific locations were chosen so as to provide information on approaches of each particular type.

The data collected included information regarding volume and turning movements, vehicular delay, vehicle arrival distributions, starting delays, and left-turn gap acceptance.

INTERSECTION SIMULATION MODEL

The simulation model developed for this study simulates two opposing approaches to an intersection and is capable of modeling any one of the following four geometric configurations: single-lane approaches with no left-turn channels, single-lane approaches with left-turn channels, two-lane approaches with no left-turn channels, and two-lane approaches with left-turn channels. The model, developed for use by the Transportation Engineering Center, Ohio State University, is referred to as the TEC model. The model is written in the specialized simulation language, GPSS/360 (General Purpose Systems Simulator).

The TEC model simulates an intersection subject to the following basic assumptions: (a) Traffic consists solely of passenger cars of similar dimensions and operating characteristics, (b) no passing or lane changing (except into left-turn channels) is permitted in the approach lanes, and (c) all vehicles enter the system with some preselected speed, such as the observed running speed.

MODEL VALIDATION

We performed a number of tests to determine the validity of the complete TEC model. Because comparison data for four different intersections were available from the data-collection phase of study, the effectiveness of the TEC model for modeling each of these intersection types was tested. The basis for the validity tests is the Kolmogorov-Smirnov one-sample test for goodness of fit. The Kolmogorov-Smirnov test is concerned with the degree of agreement between the distribution of a set of sample values and some theoretical or generated distribution. Briefly, the test is concerned with the agreement between two cumulative distributions. The point of maximum divergence D between the two distributions is determined from

$$D = \text{maximum } |F_o(X) - S_n(X)|$$

where $F_o(X)$ is a completely specified cumulative distribution function, the theoretical cumulative distribution under H_o, and $S_n(X)$ is the observed cumulative frequency distribution of a random sample of N observations. The values that have been compared between the collected data and the simulation output are the delays for vehicles passing through the intersection.

These results indicated that in all but two cases the hypothesis that the sample distribution can be reasonably thought to have come from the population distribution is accepted at the 0.05 level of significance for the 5-sec increment chosen. Both cases in which the hypothesis was rejected represent examples of low approach volumes. The two cases, however, differed considerably, and it was difficult to attribute the failure of fit to any one specific characteristic. Nevertheless, these tests do indicate that the TEC model provides adequate simulation of the types of intersections under study.

SUMMARY

The specific results of this research are related to the stated objectives of the study. The accomplishments of this research can best be discussed relative to these objectives.

The first objective was to develop a photographic data-collection system capable of recording continuous intersection traffic data, including information regarding stopped time delay of left-turn vehicles and left-turn gap-acceptance characteristics. This objective was accomplished by the development of a new camera and mirror system that uses a 35-mm robot camera with a magazine capable of recording data for 30 min at a rate of one frame/sec.

The unique data-collection system was tested in the field and was shown to be an adequate means for collecting various traffic data. Coupled with previously developed data reduction equipment, this system provided the data necessary for this study.

The second objective of the study was to develop a computer-simulation model capable of simulating the intersection of two- and four-lane roadways with and without left-turn channels. The TEC model, which can simulate these four types of intersections, was written in the simulation language GPSS/360.

The final objective of the study was to validate the simulation model, and this was accomplished by using various data collected in the field, particularly delay data.

ACKNOWLEDGMENTS

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WEAVING AREA OPERATIONS STUDY: ANALYSIS AND RECOMMENDATIONS

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Work has been completed on the first major phase of a study of the operations of weaving areas aimed at analyzing and evaluating the weaving section procedures of the 1965 Highway Capacity Manual; developing a study program that will lead to improved techniques for the analysis and design of weaving sections; and implementation of the study program to achieve the improved techniques. A hitherto unused data base of weaving section operations was available for analysis. Data collected in 1963 by the Bureau of Public Roads were restructured for computer analysis. New programs were written, and existing programs were extended and applied. In this phase, several analyses were conducted: the internal structure of the weaving capacity procedure, accuracy of the weaving and ramp capacity procedures, consistency of these procedures in specifying level of service, and specific aspects of the two ramp capacity procedures such as accuracy of lane 1 volumes. Results of these analyses indicate that quite frequently the predicted level of service differs from the actual level of service. For basic weaves and ramp weaves, the current weaving capacity procedure is as likely to show poorer as it is better levels of service. When applied to major weaves, however, it tends to predict poorer levels of service than actually occur. For ramp weave cases, the weaving capacity method produces more accurate estimates of levels of service than does either of the ramp capacity procedures, both of which tend to predict better levels of service than those actually experienced in the field. On the basis of these analyses, a reconstitution of the weaving procedure that would be applied to both major weave and auxiliary lane-ramp weave cases is recommended. A data program was specified that would enable calibration of the recommended procedure.

•IN RECENT YEARS, urban freeway design and analysis have been an area of much interest. As such, that segment of the 1965 Highway Capacity Manual (1) dealing with problems of weaving and ramps has taken on particular significance.

In 1969, the National Cooperative Highway Research Program (NCHRP) authorized a project to analyze and evaluate the weaving area procedures presented in the Manual. That project is now in progress. This paper presents some of the prime analyses done in the first major phase of the project, the resultant recommendations, and some work in progress. This reporting corresponds to the following three defined project objectives:

1. To analyze and evaluate the weaving area procedures of the Manual by using currently (1969) available field data;
2. To develop a study program that will lead to improved techniques for the analysis and design of weaving sections; and
3. To implement the study program so as to achieve the improved techniques.

The third objective was defined by the addition of continuation phases to effect the implementation. A final report (2) on the analysis and study program is available on loan from NCHRP.

Reference is made herein to the three weaving procedures discussed in the Manual. Procedure 1 is a direct analysis of simple weaving sections, procedure 2 is the regression-based approach with nomographs, and procedure 3 is the vehicle-distribution-profile approach. The Manual recommends procedure 2 for ramp cases at levels of service A to C and procedure 3 for ramp cases at level of service D. Although not specifically recommended, procedure 3 is often applied to cases at level of service E.

There are no other recent studies on this immediate topic that utilize a broad data base, but there are studies that investigate or extend particular aspects of weaving behavior, such as the analysis of three-segment multiple weaves (3) or a study of alternate striping on a test facility (4). Another is an extensive performance study of a ramp-weave configuration during two phases of reconstruction and the base condition (5). Data were collected by aerial photography. Significant improvements in travel times and reductions in internal queuing were observed as the configuration was altered, increasing the effective weaving potential within essentially fixed dimensions. There are also important related studies of lane changing (6, 7), lane distribution profiles near entrances and exits (8), and vehicle behavior out of the vicinity of ramps (9), all of which may be of assistance in the continuing study.

DATA AND TOOLS AVAILABLE

Content and Form of Data Bases

Two major data bases were available for use in this study. The first resulted from the urban weaving area capacity study conducted by the Bureau of Public Roads in 1963 at some 40 different locations in the eastern, midwestern, and far western portions of the United States. A total of 58 experiments were conducted by use of the "lights-on" survey technique. About 70 percent of the experiments were of simple weaving sections, whereas the remainder were of multiple-weaving configurations. Subsequent to the initiation of the weaving area operation study, BPR provided data for seven additional experiments conducted around the Washington, D. C., area as pilot tests for the 1963 national survey.

The data provided for each experiment included volumes by type and by lane for each entrance and exit leg. Traffic volume counts were made for 5 out of every 6 min for periods of about 2 hours. Samples of vehicle travel times through the weaving section were taken as well. In addition, relevant geometric information was provided. The data were available only in handwritten form.

The second data base, also supplied by BPR, was the sets of values used to develop the regression curves in the Highway Capacity Manual. This information was provided in the form of punched cards.

Of the two data bases available, the former was the more valuable because it provided information on physical configurations that could be analyzed by both weaving and ramp procedures and because it was "new" data—that is, it had not been used in the development of the Manual and was expected to be extremely useful in the conduct of the study.

The thrust of the first phase research was the analysis of simple weaves, of which 908, 6-min samples were available, with 11,000 travel time measures. These were structured for computer manipulation and punched on cards. Figure 1 shows the leg and lane codes used.

Tools Available and Developed

Two programs that were of considerable use were in existence at the initiation of the project. They are the weaving and ramp capacity programs developed at the Institute of Transportation and Traffic Engineering (10, 11). Before these programs were used, they were carefully reviewed and, where applicable, modified and extended to provide additional power in analysis. Some of these modifications and extensions in the weaving capacity program included the option of using either the service volumes contained in the Manual or a set of exogenously entered values, alteration of Table 7.1 of the Manual, and addition of a test of "out of the realm of weaving." In the ramp capacity program the use of truck equivalency factors on ramp grades was incorporated.

In addition to these existing programs, a battery of new programs was developed for manipulation and analysis of the data. These included programs to read in and adjust 6-min volumes, to calculate through and weaving movements by type by 6-min periods, and to compute a variety of volume characteristics for the peak hours of each experiment. Programs were also developed to compute (for each 6-min sample) space and time mean speeds by movement. As an added output of these programs, arrays were created and put on magnetic tape that contained, for each 6-min period, the weaving and through volumes by type as well as several "speed" statistics such as number of samples, sums of travel times, and sums of squares of travel times for the through and weaving movements. These arrays served as the input to other programs created to perform sensitivity and accuracy analyses as well as to the development of a new formulation for the weaving analysis and design methodology.

AREAS OF ANALYSIS

To accomplish the first project objective, the analysis and evaluation of existing procedures, we conducted several analyses, which involved the following:

1. The internal structure of the weaving procedure (procedure 1),
2. The accuracy of each of the procedures, based on both peak-hour and short-term (6-min) data,
3. The consistency of the three procedures in specifying level of service, and
4. Specific aspects of procedures 2 and 3, such as accuracy of lane 1 volumes.

Internal Structure of the Weaving Procedure

A number of analyses were undertaken to determine the viability and rationality of procedure 1. These analyses included an examination of the specified service criteria for clarity and internal consistency and an examination of the development of the weaving chart, with consideration of a recalibration thereof. The principal results of these analyses were as follows.

1. An adequate description of the operating characteristics of a weaving section requires the specification of both a level of service and a quality of flow.
2. The relationships among speed, level of service, and quality of flow are not clearly specified by the Manual, which leads to confusion in interpretation.
3. Quality of flow and level of service are not functionally dependent on each other. The consistent relationship suggested by the Manual does not exist.
4. Separate level of service standards for weaving and nonweaving vehicles would seem to produce a more accurate description of weaving section service characteristics.
5. Geometric configuration may be a vital design factor.
6. The development of the weaving chart was based on sparse data. The k-values utilized as expansion factors were rationalized and not supported by data.
7. The range of k-values exceeds the Manual specification of 1.0 to 3.0.
8. The k-values do not relate to total weaving volume V_{wo} [measured in passenger cars per hour (pcph)] and section length L as depicted in the weaving chart. Constant k-curves do not exist as suggested in the Manual.
9. Should a valid expansion exist, it appears to be more complex, involving several parameters, than that used in the Manual, in which only the minor weaving volume V_{w2} is expanded.

Description of Service Characteristics—Although it is not clearly stated, the use of the Manual procedure requires the specification of both a level of service and a quality of flow. Consider the equation for the width of a weaving section:

$$N = [V_T + (k - 1)V_{w2}]/SV \qquad (1)$$

where

N = number of lanes in section,
V_T = total volume in section,

k = expansion factor,
V_{w2} = minor weaving volume, and
SV = service volume.

The length of the weaving section and the k-value used in the width equation are determined by entering the weaving chart with a specified weaving volume (in pcph) and quality of flow. SV is selected from Table 9.1 in the Manual (for freeways) and is dependent on a specified level of service.

Most properly, quality of flow relates to the speed of weaving vehicles alone. Level of service describes the speed of all vehicles combined. Neither of these can adequately describe the operating characteristics of a weaving area. Inasmuch as quality of flow relates only to weaving vehicles, it may not be used alone to describe a section containing both weaving and nonweaving vehicles. Level of service treats collectively two flows with often widely differing characteristics and effectively conceals such differences. Only when both are specified is a complete picture drawn. Even this, however, produces an awkward, indistinct description.

Speed Criteria—There are several problem areas that create a degree of confusion in the speed-service relationships detailed in the Manual. The first of these involves the use of operating speed as a criterion. Strictly defined, operating speed is the maximum speed at which a car may travel under prevailing traffic and roadway conditions without at any time exceeding the design speed. Most properly, this parameter is measured with a test vehicle observing sample vehicles. From such a sample speed distribution, such items as 85th percentile speed, median speed, and space mean speed may be determined. None of these corresponds directly to operating speed, although they may be used to estimate it. Of greater importance is the fact that such sample data were used to calibrate Manual procedures and were collected in the 1963 urban weaving area capacity study. It is of extreme importance that sample data be accurately segregated into specified service standard categories. Some of the analyses reported herein required such stratification by service categories. For these analyses, space mean speed rather than operating speed was used.

The stated speed criteria are ambiguous to a large degree. The specification of quality of flows I and II states that speeds of 50 mph or more and 45 to 50 mph respectively "are attainable." Whether these speeds refer to all vehicles, weaving vehicles, or nonweaving vehicles is not clear. It is assumed that only weaving vehicles are included, as criteria for quality of flows III, IV, and V (40 to 45, 30 to 35, and <30 mph respectively) specifically refer only to these.

Level of service criteria are similarly unclear, with the Manual suggesting that speeds in weaving sections for a given level of service be 5 to 10 mph lower than those on similar sections of open highway. Open highway standards are taken from Table 9.1 of the Manual (for freeways) or corresponding tables. Because these tables refer to the average speed of all vehicles, it is assumed that all vehicles are included in the application of adjusted standards to weaving areas.

Also of concern is the discontinuity in both level of service and quality of flow criteria for speeds of 35 to 40 mph. Several of the analyses reported herein required determinations of level of service and quality of flow, so standards were adjusted to provide continuous boundaries. For level of service in weaving areas, 10 mph was deducted from open highway standards. The standards used are given in Table 1.

Quality of Flow and Level of Service Relationships—Table 7.3 of the Manual details a relationship between level of service and quality of flow, which is presumed to be consistent. However, when we consider the parameters that determine each, it can be seen that no consistent dependence exists. Analytically, quality of flow as determined by the weaving chart depends on the weaving volume and the length of the segment. Level of service depends on the service volume, which is found by dividing the total expanded volume by the number of lanes. Although these parameters are loosely related, it can be seen that specification of a quality of flow does not automatically yield a level of service or vice versa. The full range of quality of flow-level of service combinations is theoretically feasible, and conditions actually occurring are not restricted to those combinations shown in Table 7.3 of the Manual.

These observations are supported by data from the 1963 BPR study. If actual qualities of flow and levels of service are identified by sample speeds, 15 of 45 experiments reveal combinations not indicated in the Manual. Because the space mean speed (SMS) of all vehicles numerically includes the SMS of weaving vehicles, even those experiments that conform to the Manual may be more indicative of a computational dependence on rather than a real interrelationship between flows.

The unrestricted nature of the level of service-quality of flow relationship can be seen in both analysis and design. Consider, for example, a weaving configuration long enough to be "out of the realm of weaving." Such a section may conceivably operate at quality of flow I as analytically determined by V_{we} and L but will experience the full range of levels of service based on total volume fluctuations. Because of the great length of such a section, weaving volumes may never be high enough to deteriorate the quality of flow. Although analytic determinants may indicate, for example, quality of flow I and level of service D, the high weaving speeds predicted for quality of flow I will not be achieved, as total volumes restrict the entire operation to level of service D.

In design, a similar situation is encountered. When the width equation $N = [V_T + (k - 1)V_{w2}]/SV$ yields fractional results, additional length may be provided to reduce N to the nearest whole number. In this way, a more economical design is achieved. However, as the length is increased, a better quality of service is attained. Level of service, on the other hand, remains unchanged.

It can be seen that the analytic relationship between level of service and quality of flow is unrestricted. In the use of these measures in analysis, it is necessary to determine which of the two measures gives a more realistic description of operations. In general, this will be the "worse case," as in the example above where quality of flow I could not actually be achieved due to the low level of service. In design, because the general design level of service for a given facility is of primary interest, the quality of flow for weaving areas should be as good as or better than the design level of service.

A Recommended Descriptor of Service—It was pointed out that no functional analytic relationship exists between quality of flow and level of service. It was also stated that actually occurring values do not conform to the relationship predicted by the Manual and that the inclusion of all vehicle speeds in the level of service description may mask significant differences between weaving and nonweaving flows. Such differences often occur, and, because they do, it would appear that separate levels of service for weaving and nonweaving vehicles would be more descriptive of actual operating conditions.

Geometric Effects—It is valuable to investigate why drastic differences in weaving and nonweaving speeds occur in some cases and not in others. It would appear that geometric configuration is a major factor. Data given in Table 2 illustrate that speed differences occur most often on ramp-weave sections and that the differences are generally larger than those observed for other configurations. In the ramp-weave configuration, weaving vehicles are more or less restricted to the auxiliary lane and the shoulder lane regardless of the total number of lanes provided. Additional lanes in ramp-weave sections will be used primarily by nonweaving vehicles. Where total width is excessive, weaving vehicles may operate at low speeds in two lanes while outer flows travel at considerably higher speeds in other lanes. Major weaves, which vary widely with configuration, are generally not as restrictive. This is shown in Figure 2.

The number of lanes actually occupied by weaving vehicles was computed for each experiment from the 1963 study data. In no ramp-weave case was 2.0 exceeded, whereas in the majority of major weave cases weaving vehicles occupied more than 2.0 lanes. This result supports the hypothesis, but it cannot be viewed as conclusive because the major weave cases entailed generally higher weaving volumes than ramp-weave cases and would normally be expected to occupy more lanes. However, this result, coupled with the frequent occurrence of speed differences in ramp-weave cases, indicates that the hypothesis has merit.

Thus, the Manual procedure of computing total lane requirements may be misleading. Lane requirements for weaving and nonweaving flows should be separately computed so that a configuration allowing appropriate lane use may be designed.

Development of the Weaving Chart—The original data and rationale behind the weaving chart have not been documented and are not available for study. However, certain facts concerning the development of the chart are known and may be commented on.

Figure 1. Standard coding for weaving data.

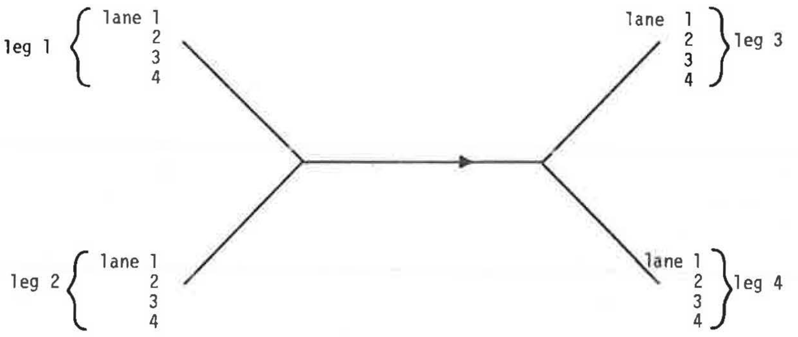


Table 1. Service criteria.

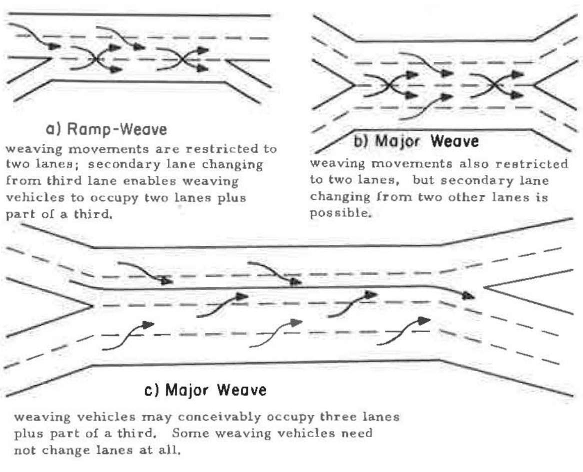
Level of Service	SMS of All Vehicles: On Freeways	SMS of All Vehicles: In Weaving Areas	Quality of Flow	SMS of Weaving Vehicles
A	≥60	≥50	I	≥50
B	55 to 60	45 to 50	II	45 to 50
C	50 to 55	40 to 45	III	37.5 to 45
D	37.5 to 50	27.5 to 40	IV	30 to 37.5
E	30 to 37.5	20 to 27.5	V	<30
F	<30	<20		

Table 2. Nonweaving vehicles having SMS different from those of weaving vehicles.

SMS (mph)	Number of Vehicles: RA Section[a]	Number of Vehicles: M and CD Sections[b]	Number of Vehicles: All Sections
>5	1	2	3
±5	10	17	27
5 to 10	0	4	4
10 to 15	2	1	3
>15	4	0	4

[a] Ramp weave with auxiliary lane formed between consecutive on- and off-ramps.
[b] Major weave with at least three legs having more than two lanes and weaving sections on collector-distributor roadway.

Figure 2. Weaving movements as affected by configuration.



The original weaving chart of the 1950 Highway Capacity Manual involved three plots on a V_{we} versus L field: one for maximum possible capacity, one for 30-mph operating speed, and one for 40 mph. These three curves were based on field data and were adjusted slightly in 1957 (12). These three curves became curves III, IV, and V in the 1965 Manual. The original equation for width was similar to the present one but contained a constant expansion factor of 3.0 rather than a variable k based on V_{we} and L. The 3.0 expansion factor was rationalized on the basis of approximate gap size necessary to execute a weaving maneuver and not on observed data. By the time the 1965 Manual was being formulated, limited amounts of data permitted estimation of curve I for "out of the realm of weaving." For this curve, the expansion factor was logically 1.0. This left the problem of providing a smooth expansion transition from 1.0 below curve I to 3.0 above curve III. The intermediate curves of the 1965 Manual are the result of a constructed transition.

Therefore, whereas the length-weaving volume relationships depicted by curves I, III, IV, and V of the 1965 Manual weaving chart are based on limited amounts of data, the k-factor expansion mechanism has not been subjected to calibration.

The Range of k-Values—Freeway experiments of the BPR study were used to calibrate and verify the constant k-curves of the weaving chart. With the use of the width equation, with all values except k known, k may be computed as

$$k = [N(SV)/V_{w2}] + [1 - (V_T)/(V_{w2})] \tag{2}$$

where the terms are as defined. Service volume is given in Table 9.1 of the Manual for each level of service as identified by the SMS of all vehicles (the speed criteria of Table 1 are used).

A problem arises in that only integer values of N are observed, whereas in design fractional values may be obtained. Thus, an error from rounding off results, causing inflated values of k to appear. These errors arise, however, because SV is treated as a step function, with one value for a range of speeds. In actuality, all lanes are used. If a fractional part of a lane has been added to the design computation, speeds slightly higher than the minimum for the level of service used will result. Therefore, if the values of speed detailed in Table 1 and the SV values of Table 9.1 in the Manual are viewed as threshold values and a straight-line interpolation between values is used, a value of SV based on the exact observed speed may be selected and the round-off error eliminated.

If step function SV values are used, it is possible to compute the maximum round-off error for each experiment (2). The analysis presented is more easily manipulated and interpreted. This was done, and k-values were computed. For 16 ramp-weave cases, k took on 3 values above 3.0 and 4 below 1.0. Of 19 major weaves, 8 values were significantly above 3.0, and one was below 1.0.

Values below 1.0 are disturbing, inasmuch as it does not seem feasible that a vehicle among V_{w2} is equivalent to less than 1.0 other vehicle and certainly does not occupy negative space. Such values may be the result of unusual geometric conditions, such as sharp loop ramps, that exist at one of the sub-1.0 experiments or extra wide lanes that exist at another. In this latter case, a 72-ft roadway was striped for 5 lanes although vehicles had room to form 6. Sampling errors may have also influenced these values.

Despite this concern, the upper limit of 3.0 has most certainly been shown to be false, inasmuch as 11 of 26 computed k-factors are beyond this limit. The calibration does not, however, clearly indicate or suggest any other upper limit on k.

The Relationship of k to V_{we} and L—The k-factors were plotted on the V_{we} versus L field (Fig. 3) in an attempt to reestablish the constant k-curves of the 1965 Manual weaving chart. The plot clearly shows that no such constant k-curves exist and that the relationship among k, V_{we}, and L is not as is depicted in the Manual.

The Expansion Concept—Before we discarded the basic idea of an equivalence expansion mechanism, a number of possible alternatives were examined. Two additional sets of expansion factors k_{V_w} and $k_{V_{w1}}$ were computed based on expansion of the entire weaving volume V_w and the larger weaving volume V_{w1}. These were plotted on the V_{we}

versus L field, and, as in the case of the k-factors, no constant value curves were formed. However, all three expansion constants, k, k_{V_w}, and $k_{V_{w1}}$, exhibited promising correlations when plotted versus the ratios V_w/V_T and V_{w2}/V_{w1}. Although not conclusive, these results suggest two things about the true expansion mechanism: Expansion of both V_{w2} and V_{w1}, perhaps individually in an additive fashion, should be considered; and the expansion value seems to depend on both the percentage of weaving vehicles in the traffic stream and the split between V_{w1} and V_{w2}. A predictive mechanism for k, therefore, should involve both parameters. It is concluded that a valid expansion model would be far more complex than that used in the 1965 Manual. The data at hand are not sufficient to investigate possible forms. Because of the difficulties involved in collecting such data and the difficulties involved in formulating such a model, it appears that development of a design procedure that does not directly involve equivalence expansion would be advisable.

Analysis of the Accuracy of Manual Procedures

It was decided, where possible, to test the accuracy of all three procedures in predicting actual levels of service.

A problem immediately arises because the speed-level of service relationships that must be used to identify field levels of service differ in Chapters 7 and 8 of the Manual. Procedures 2 and 3, from Chapter 8, use the relationships of Table 9.1 directly, whereas procedure 1, from Chapter 7, specifies a deduction of an ambiguous 5 to 10 mph from these standards. In the internal analysis of the weaving procedure 1, the authors used the 10-mph deduction for consistency. For accuracy, a number of alternatives were tested, including one suggested by a principal in the development of Chapter 7 of the Manual. Results indicated that this latter specification correlated best to predicted levels of service; therefore, only results for this case are reported. The speed-level of service relationships used in the accuracy analysis are given in Table 3.

The problem that in the Manual level of service C means different standards depending on the procedure used must be kept in mind when the results of the accuracy analyses are considered. The analysis considered basic weaving sections (in which all traffic weaves), ramp-weave cases, and major weave cases separately. Only in the case of ramp weaves may all three procedures be applied and compared. Only procedure 1 is used in other cases. Data from the 1963 BPR study were utilized for both peak-hour data and individual 6-min periods. The results of the analysis are given in Table 4.

The following conclusions may be drawn from these results:

1. The accuracy of level of service predictions by procedure 1 is highest for basic weaving sections, followed by ramp weaves and major weaves. Accuracy of the procedure is generally poor, inasmuch as less than a third of all experiments were accurately predicted. Use of operating speed would have further degraded the accuracy.
2. For basic weaves and ramp weaves, the majority of errors are by a single level of service, with no trend toward being poorer or better than actual values for procedure 1. When applied to major weaves, procedure 1 tends to predict levels of service poorer than those that actually occur.
3. Although the Manual recommends the use of procedures 2 or 3 for ramp-weave cases, procedure 1 produces more accurate estimates of level of service.
4. Level of service predictions for ramp-weave cases by procedures 2 and 3 tend to be better than actual field conditions.

The accuracy of procedures 2 and 3 as regards ramp-weave cases was further investigated. These procedures depend on the prediction of lane 1 volumes in advance of ramps. Accordingly, lane 1 volumes were computed by procedures 2 and 3 immediately in advance of the on-ramp and were compared to actual volumes. Although the Manual recommends procedure 2 for cases of levels of service A to C and procedure 3 for levels of service D and E, both methods were applied to all experiments where possible.

The accuracy of procedure 2 for levels of service A to C is shown in Figure 4. Differences between computed and observed ranged from 6 to 24 percent with an average difference of 15 percent. The sample size, however, was only 4, and definitive conclusions may not be reached.

Figure 3. Computed k-factors on a weaving chart.

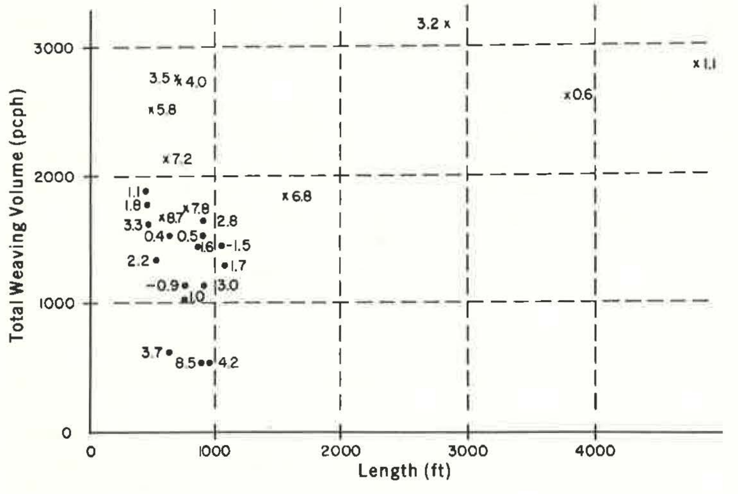


Table 3. Service criteria for accuracy analysis.

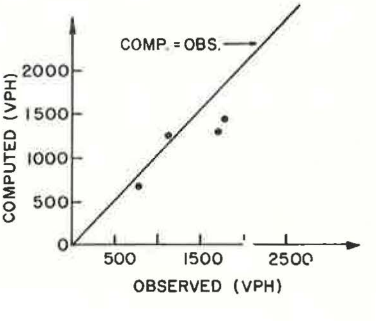
	SMS of All Vehicles			SMS of All Vehicles	
Level of Service	Procedures 2 and 3	Procedure 1	Level of Service	Procedures 2 and 3	Procedure 1
A	≥60	≥50	D	37.5 to 50	25 to 37.5
B	55 to 60	45 to 50	E	30 to 37.5	15 to 25
C	50 to 55	37.5 to 45	F	≤30	≤15

Table 4. Percentage of difference between actual and predicted levels of service.

			Level of Service				
Procedure	Weave Section	Sample Period	Same	1 Level Better	1 Level Poorer	>1 Level Better	>1 Level Poorer
1	Basic	Peak-hour[a]	50	16	16		16
		6-min	30	34	17	8	11
	Major	Peak-hour	27		69	4	
		6-min	21	7	43	4	25
	Ramp	Peak-hour[b]	35	23	35	7	
2	Ramp	Peak-hour[b]	23	41	12	24	
3	Ramp	Peak-hour[b]	20	40		40	

[a]Sample size only six experiments.
[b]Six-min data not shown for brevity; results are similar.

Figure 4. Computed versus observed lane 1 volume for levels of service A to C, procedure 2, peak-hour data.



Twenty experiments were determined to be in levels of service D and E. When lane 1 volumes were computed by procedure 3, the differences between observed and computed values ranged from 1 to 70 percent with an average of 25 percent. As shown in Figure 5, most errors involve computed values lower than actual values, a serious condition that may result in inadequate designs.

Thirteen of the 20 levels of service D and E cases were also examined by procedure 2. Differences between observed and computed lane 1 volumes ranged from 1 to 43 percent, with an average of 17 percent, a distinct improvement over procedure 3 results (Fig. 6). Despite the Manual specification of procedure 3 for these cases, lane 1 volumes were more accurately predicted by procedure 2 in 10 of 13 cases.

It should be noted that procedure 3 most properly applies only to level of service D. It is prescribed for checking a given ramp-weave segment or ramp to see whether it meets the requirements for the high-volume threshhold of level D. The accuracy analyses referenced herein did in fact do this. When the criteria for level D are not met, level E was assumed. The method was extended to include a check against Table 8.1 (of the Manual) level E checkpoint values to determine whether a level F condition was indicated.

These results show that procedure 2 produces more accurate levels of service predictions than procedure 3 for ramp-weave cases with auxiliary lanes, even for cases of levels of service D and E. Six-min data were used to further examine the accuracy of procedure 2 for all levels of service. An average difference between observed and computed lane 1 volumes of 19 percent was obtained. A general trend toward decreasing accuracy as length of the section increases was noted. The angle of approach at on-ramps was also investigated, but results indicated that it had little effect on the accuracy of lane 1 volume predictions in the normal range of 1 to 6 deg.

The accuracy of Figure 8.22 of the Manual, which predicts the percentage of trucks in lane 1, was also tested. Differences between observed and actual values ranged from 1 to 37 percent with an average of 13 percent. Particularly in the case of eight-lane freeways, the results predicted by the Manual are markedly different from a regression line fit to the actual data. This is shown in Figure 7. Although the differences noted for four- and six-lane freeways are not as drastic, Figure 8.22 of the Manual does not appear to accurately represent the relationship between freeway volume and percentage of trucks in lane 1.

Consistency of Procedures 1, 2, and 3 in Specification of Level of Service

The consistency of the three procedures in specifying levels of service was examined by comparing predictions for ramp-weave cases of the 1963 BPR study. To obtain a comparison over a wider range of levels of service, we constructed and analyzed a range of cases. The results of the analysis indicate that procedure 1 yields level of service estimates poorer than procedures 2 and 3 for relatively short or wide sections and better levels of service than procedures 2 and 3 for longer, narrower sections. These general results, however, must be viewed in light of the fact that level of service criteria differ for procedure 1 and procedures 2 and 3. Because of this problem, the results of the accuracy analyses must be viewed as the more meaningful.

Adjustments to Current Manual Procedures

The results of the analyses reported earlier point out the need for a new weaving methodology. Work is progressing along these lines and is described later. An improved algorithm is being developed; several modifications in the use of current procedures can improve their accuracy.

1. Level of service criteria for procedure 1 (Manual, Chapter 7) have been shown to be unclear. The accuracy analysis showed that the following standards resulted in the best correlation to predicted values of the alternatives tested. It is therefore recommended that the level of service criteria given in Table 5 be adopted. (Note that in this table space mean speed rather than operating speed is used as a correlate.)
2. The quality of flow-level of service combinations shown in Manual Table 7.3 suggest that these are the only feasible combinations. This has been shown to be weak,

in both actual cases, based on observed speeds, and analytically, in the case of predicted values based on known volumes and geometric factors. In design, a quality of flow that will not interfere with the maintenance of the design level of service under design hour volumes must be selected. In analysis, prediction of incompatible level of service and quality of flow indicates that the poorer condition will most likely prevail. Because design level of service for an entire facility is of primary importance, the level of service should be the controlling measure in well-planned sections.

3. Geometric configuration appears to have a marked effect on the operation of a weaving section. Certain configurations have been shown to restrict weaving vehicles to a portion of the roadway, regardless of total width. For this reason, lane requirements for weaving and nonweaving vehicles should be separately computed and considered for suitability. The Manual width equation may be modified:

$$N_w = N_{weaving} = (V_{w1} + kV_{w2})/SV$$

$$N_{nw_1} = N_{nonweaving_1} = (V_{o1}/SV) \; N_{nw_2} = N_{nonweaving_2} = (V_{o2}/SV)$$

where

V_{wi} = weaving volume i,
V_{oi} = outer volume i,
SV = service volume,
k = expansion factor,

and N_j designates number of lanes (which may be fractional) for purpose j. Note that N_{nw_1} and N_{nw_2} are computed separately and must be provided on opposite sides of the weaving lanes. In this way, a configuration that provides an adequate number of total lanes and that permits weaving vehicles to use the required number of lanes may be designed. The Manual specifies that ramp-weave sections with auxiliary lanes should be treated as suggested in the procedures given in the Manual, Chapter 8. The accuracy analysis has shown that the Chapter 7 weaving procedure is more accurate in these cases despite its weakness. It is therefore recommended, as an interim measure, that such cases be analyzed according to Chapter 7 and not Chapter 8. Service criteria outlined in the first item apply.

RECOMMENDATIONS

On the basis of the analyses conducted in the first major phase, it was recommended that the weaving procedure be reconstituted, that it incorporate both major weave and auxiliary lane cases, and that it be macroscopic in approach. It is further recommended that lane balance and geometric capability be explicitly considered to be of prime importance in the procedure. Given that values of certain macroscopic variables are computed (weaving width N_w and length L specifically), it is essential that the configuration be such that these may in fact be provided. Conversely, the specification of a configuration effectively determines the range of N_w that is realizable in a given length. As evidenced by Gafarin (5), a change of configuration within fixed dimensions can significantly affect the weaving capability. Configuration as it influences weaving is a subject of on-going research.

To realize these general recommendations, we specified a study program. The prime points of that program are as follows.

1. Further data efforts in regard to weaving sections should be devoted to three areas: collection of the data for the calibration of the reconstituted procedure, collection of some detailed supplemental data to enhance the engineer's understanding of the basic mechanisms of weaving, and synthesis of existing data banks and research related to weaving.
2. The principal data collection effort should be devoted to the calibration of the reconstituted procedure at levels of service B and C particularly, the existing base having little such data, and over those lengths not well represented in the existing base, even at levels of service D and E.

3. The speed measure used in the reconstituted procedure should be space mean speed as is the case with the AASHO policies, with travel time samples used to estimate SMS.
4. The data collection generally should be done by ground-based time-lapse photography, with filming done for 4 to 6 consecutive hours so as to observe not only the various levels of service but also the transitions from one to another.
5. Based on the data collected, the model for the weaving design-analysis procedure should be revised and calibrated as necessary.

This study program has been accepted by NCHRP, and it is now being executed.

It should be noted that some work has been done on the form and equations of the reconstituted procedure (2). This is an outline of probable structure and is not suitable for promulgation at this time. It does, however, propose (a) explicit consideration of the section configuration in its calibration and use, (b) a particular mathematical form involving weaving parameters (volume, length, width, and minor-to-total weaving ratio), (c) functional constraints on length-width combinations and on minimum length, and (d) an investigation of the number and range of plateaus (levels of service) that exist in a weaving situation.

The research under way is considering a full range of section lengths, including rather short sections. These will provide some important basic knowledge of weaving intersections and a basis for the analysis of existing sections. They may also provide information for a design of last resort in some urban areas, but it will be most important that these not be able to be interpreted as desirable design.

CONDUCT OF DATA BASE PHASE

As part of the study program, it was specified that data would be collected at 16 major weave or ramp-weave sites and at one multiple-weave configuration. All data are to be of the recommended 4 to 6 hours' duration, with the possible exception of the multiple weave, the duration of which is to be controlled by cost. All sites are located in the northeast United States, no discernible variation with geography having been observed in the existing data base.

In general, data are to be collected by fixed-position time-lapse photography. A limited number of sites (the longer major weaves) may require a hybrid collection mode that includes input-output license plate recording and tracing vehicles by calibrated camera from a vantage point that would not permit fixed-camera photography.

The fixed-position camera system for sites visible by one camera is generally a 16-mm Beaulieu time-lapse camera with a 200-ft magazine, intervalometer, and a 50-mm Angenieux zoom lens with a split-image adaptation. This adaptation permits a calibrated timer to be shown on the film. For sites that require two cameras for adequate coverage, a super-8 Minolta (Autopak 806) is used as the second camera. This camera is also equipped with intervalometer, zoom lens, and split-image adaptation. Sample photographs of two sites are shown in Figures 8 and 9. These are two 990-ft major weaves on the Cross-Bronx Expressway near the George Washington Bridge in New York City and a 700-ft ramp weave on the Kensington Expressway in Buffalo, New York, respectively. The first site was filmed with the super-8 system, which was backing up the 16-mm system on that site. The calibrated timer is clearly in view in both cases.

The data are being reduced by teams of two who either observe and trace each vehicle (or a sample thereof) or count volumes and then trace vehicles for travel time samples, depending on the type of site and its outer flows. An L&W photo data analyzer model 224-A is being used for 16-mm films, and a Kodak model MFS-8 super-8 stop-action projector is being used for the super-8 films. The calibrated timer is read and recorded appropriately. All data are recorded on forms from which keypunching can easily be done. Keypunched data are checked for common recording and punching errors by special computer programs.

ACKNOWLEDGMENTS

The authors wish to acknowledge the participation and assistance of Jack E. Leisch, who served as a consultant on this project, and the support of NCHRP. This study is

Figure 5. Computed versus observed lane 1 volumes for levels of service D and E, procedure 3, peak-hour data.

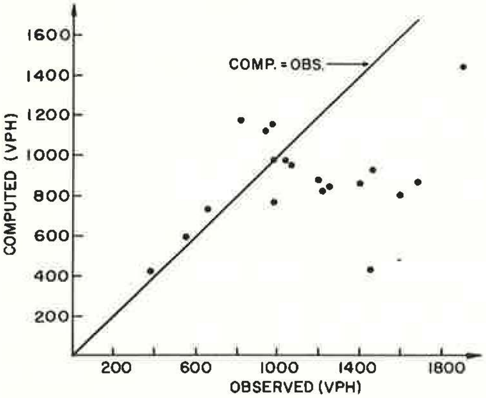


Figure 6. Computed versus observed lane 1 volume for levels of service D and E, procedure 2, peak-hour data.

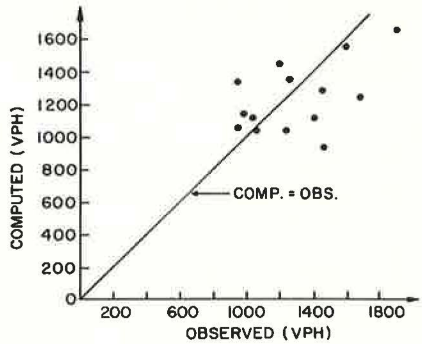


Figure 7. Eight-lane freeway (all experiments).

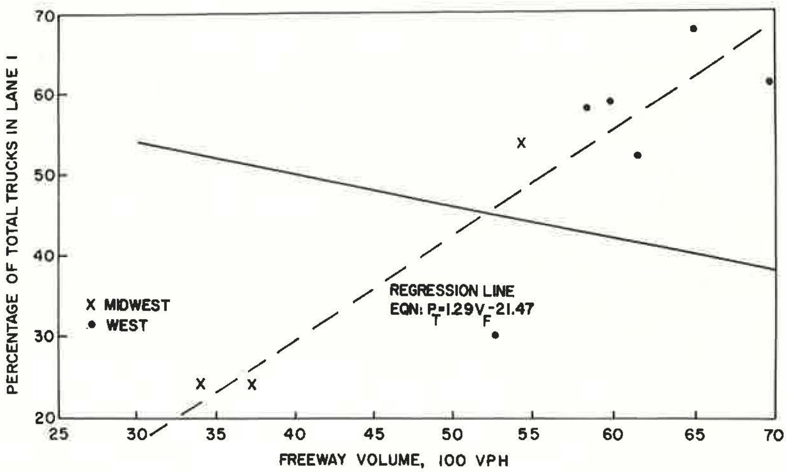


Table 5. Recommended interim service criteria.

Level of Service	SMS of All Vehicles	Level of Service	SMS of All Vehicles
A	≥50	D	25 to 37.5
B	45 to 50	E	15 to 25
C	37.5 to 45	F	≤15

Figure 8. Data taken on Cross-Bronx Expressway, New York City.

Figure 9. Data taken on Kensington Expressway, Buffalo.

drawn from a National Cooperative Highway Research Program project. The opinions and findings expressed or implied in this paper are those of the authors and not necessarily those of the Highway Research Board, the National Academy of Sciences, the Federal Highway Administration, the American Association of State Highway Officials, or the individual states participating in the National Cooperative Highway Research Program.

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DISCUSSION

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During the several years that have elapsed since the 1965 Highway Capacity Manual was published, I have had the opportunity to work extensively with the procedures recommended for computing the service volumes and levels of service for ramp junctions and weaving sections. This involvement has been primarily as a user and as an instructor in several workshops and seminars aimed at teaching the use of the Manual. Therefore, I have reviewed and would like to comment on the paper prepared by Pignataro and his associates from the viewpoint of a user and not a theoretician.

To the average technical employee of a highway engineering organization, be it a government agency or a consulting firm, the two main criteria for judging an analysis or computation procedure are the following: Is it straightforward and easy to understand and use; and are the answers derived sufficiently accurate to satisfy the requirements of the overall project involved? The technical employee making the analysis is not usually interested in the theoretical basis for the procedure and, generally, does not have the time to analyze the underlying principles even if he is interested. With these thoughts in mind, I would like to comment on a few of what I consider to be the major findings, conclusions, and recommendations of the paper.

A substantial portion of the paper is devoted to an analysis of the internal structure of the weaving procedure described in Chapter 7 of the Manual to determine the viability and rationality of the procedure. Of the nine major conclusions reached as a result of this analysis, only one would appear to be of major concern to the user.

The paper indicates that the relationships among speed, level of service, and quality of flow are not clearly specified by the Highway Capacity Manual. Of major concern is the use of the "operating speed" to define the quality of flow and indirectly the level of service. The fact that the operating speed is not a directly measurable quantity has led to problems of not only analysis of weaving sections but also the free flow analysis of freeway, expressway, and two-lane highway sections. The use of a measurable parameter such as the space mean speed would certainly eliminate much of the confusion that has been generated through the use of the term "operating speed."

The discontinuity in level of service and quality of flow criteria at speeds of 35 to 40 mph, which exists in both Chapter 7 and Chapter 9 of the Manual, have also been of concern to many users. Although the discontinuity has not led to any particular user problems, it has resulted in a certain amount of apprehension by some users who then question the creditability of the whole procedure.

The results of the analysis of the accuracy of the three weaving procedures should certainly be encouraging and welcomed by most persons involved in analyzing weaving sections. I believe that the vast majority of organizations and individuals involved in the analysis of weaving sections are using the Chapter 7 procedure for analysis of all types of weaving sections of all levels of service. Although the basic decision to disregard the recommendations of the Manual were, I believe, based on the complexity of the procedures described in the chapter on ramps, the conclusion that procedure 1 produces more accurate estimates of service level although the Manual recommends the use of procedures 2 or 3 for ramp-weave cases supports the decision.

The results of the accuracy analysis are also encouraging from a second point of view. The Chapter 7 procedure accurately predicted the level of service in 50 percent of the basic weaving section experiments and was within one level of service in 80 percent of the cases. For ramp-weave sections and major weaves, the Chapter 7 weaving procedure came within one level of predicting the actual level of service in over 90 percent of the cases analyzed. Because the actual level of service was determined by calculating the space mean speed of all vehicles, it is possible that a number of the cases that the Chapter 7 procedure predicted as being one level of service too high or too low missed the mark by only 1 or 2 mph. If this is the case, I believe that the results are even more encouraging than might be believed at first glance.

As a result of the various analyses made, the paper recommends several modifications in the current procedures. Table 5 includes recommended service criteria utilizing the space mean speed of all vehicles rather than the operating speed, eliminates the discontinuity at speeds of 35 to 40 mph discussed earlier, and should close the creditability gap that exists in the minds of some people. The use of space mean speed rather than operating speed should increase the usefulness of the procedure as a tool for evaluating operations on existing roadways.

The second recommendation contains guidelines for resolving the limited relationship between level of service and quality of flow specified in Table 7.3 of the Manual. This recommendation is relatively obvious and is, I believe, being followed by most individuals involved in analyzing weaving sections.

The paper also recommends that the number of lanes required for weaving vehicles be calculated separately from the number of lanes required for nonweaving vehicles. Whereas the total number of lanes thus calculated is identical to the number calculated by using the Chapter 7 weaving procedure, the analyst will be better able to visualize the need for the lanes, the geometric configuration required, and how the section of roadway will operate. Properly utilized, this information could result in improved geometric designs; improperly used, the design would be the same as would result by utilizing the Chapter 7 procedure without modification.

The final recommendation is that the Highway Capacity Manual Chapter 7 weaving procedure be used to analyze ramp-weave sections with auxiliary lanes in place of the Chapter 8 procedures. As was previously indicated, most organizations are currently following the Chapter 7 procedure, and this recommendation tends to support their action.

In summary, I heartily support all four adjustments to the Highway Capacity Manual procedure and believe that they would be an improvement over the Manual when judged by the user's criteria of simplicity and accuracy.

AUTHORS' CLOSURE

The authors are pleased that Sullivan found the paper satisfying and thank him for his detailed attention in the review. We offer only a few comments on his discussion.

It is agreed that the average user of a computation procedure would not be concerned with its theoretical aspects in day-to-day applications, nor should he be. He should, however, satisfy himself at some time that it is a sound and rational procedure. Moreover, those providing him with the procedure as a tool should ensure this. This is the intent of our analysis of internal structure.

The possibility that a significant number of cases were out of the computed (by the Manual) level of service by only 1 or 2 mph was considered and was not a significant occurrence. The general problem of level of service in a weaving section by speed or volume categorization or both, however, is a substantial one and is being considered in the continuing project research. In particular, the redefinition of the number and boundaries of levels within a weaving section and the existence of a critical region in which speed performance is not simply related to volume and geometrics are being considered.

The fact that separate computation of the lanes required on each side for nonweaving vehicles is numerically identical to the existing Chapter 7 procedure is quite true. Sullivan correctly points out, more clearly than the original text, that the advantage is in properly assessing the requisite configuration.

A CAPACITY ANALYSIS TECHNIQUE FOR HIGHWAY JUNCTIONS

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ABRIDGMENT

• LINEAR PROGRAMMING is a general form of applied mathematics in which a linear function (called the objective function) of a set of variables is maximized or minimized (depending on the nature of the problem) subject to a set of linear constraints. The procedure used to solve the linear programming problem will determine the values of the variables that maximize or minimize the objective function and that do not violate any of the linear constraints.

To supplement this verbal description, consider the following mathematical statement of the general linear programming model. The objective function takes the form

$$W_1V_1 + W_2V_2 + \ldots + W_nV_n$$

where V_i is a set of variables for which the values are sought and W_i is the value weights of each of the variables. This is the linear function that is to be either maximized or minimized. Next, a set of constraints is developed that define the numerical range from which the values of the variables are taken. An example of such a constraint is

$$A_{i1}V_1 + A_{i2}V_2 + \ldots + A_{in}V_n \; \{<, =, >\} \; C_i$$

where C_i is a constant and the A values are the value weights of each of the variables. The general form of the linear programming problem can be stated as follows:

$$\text{Maximize or minimize } W_1V_1 + W_2V_2 + \ldots + W_nV_n$$

subject to

$$\begin{aligned} A_{11}V_1 + A_{12}V_2 + \ldots + A_{1n}V_n \; &\{<, =, >\} \; C_1 \\ A_{21}V_1 + A_{22}V_2 + \ldots + A_{2n}V_n \; &\{<, =, >\} \; C_2 \\ &\vdots \\ A_{m1}V_1 + A_{m2}V_2 + \ldots + A_{mn}V_n \; &\{<, =, >\} \; C_m \end{aligned}$$

This is a brief discussion of a rather complex form of mathematics. However, because of the widespread applications of linear programming, sufficient documentation is easily available.

INTERCHANGE CAPACITY ANALYSIS MODEL

General Approach

A linear programming model provides a rather effective means of determining the capacity of any type of interchange, regardless of the complexity of the geometric configuration. The capacity is defined as a maximum volume capable of entering the interchange without causing the capacity of any geometric element of the interchange to be exceeded. Because the capacity is the maximum number of vehicles that can use the interchange, the objective function of the model is to be maximized subject to the geometric characteristics of the interchange and vehicular distribution through the interchange.

Development of the Objective Function

The total number of vehicular movements that may occur at an interchange is a function of the number of approaches. The function is

$$x = n\,(n - 1)$$

where x is the maximum number of possible movements and n is the number of approaches. Therefore, for a four-leg interchange, with all movements permitted, there are 12 possible movements. Thus, the variables for the objective function represent the volume of each of the 12 movements approaching the interchange. Each possible movement must be explicitly defined. Because the volume entering the interchange is to be maximized, the objective function for a four-leg interchange capacity problem is

$$\text{Maximize } V_1 + V_2 + V_3 + V_4 + \ldots + V_{12}$$

or

$$\max \sum_{i=1}^{12} V_i$$

Because each variable in the objective function represents an individual movement, a weight value of one is associated with each variable. To determine the maximum volume capable of entering the interchange requires that this function be maximized subject to various geometric and vehicular distribution constraints.

Development of the Constraints

There are two types of constraints that must be met. The first set of constraints relates to the capacity of each of the interchange elements. Each of these constraints is related to a particular geometric element of the interchange and states that the volume using the element must not exceed the capacity (or service volume) of the geometric element. The specific values of the capacity constraints actually define the geometric characteristics of the interchange to be analyzed. The number of approach lanes, the number of lanes for a particular movement, and the signalization must be defined with respect to their individual capacities. Data shown in Figure 1 indicate that, if movements V_3 (right-turn volume from the west) and V_7 (left-turn volume from the east) must both use the same entrance ramp that has a capacity C_3, the following constraint must hold true:

$$V_3 + V_7 \leq C_3 = f\ (\text{number of lanes})$$

where C_3 is the capacity of the entrance ramp. Another example of physical constraint is

$$V_1 + C_2 + V_3 \leq C_{12} = f\ (\text{number of lanes})$$

where C_{12} is the capacity of the west approach.

This constraint must not be violated because these three movements enter the interchange from the west approach. If signalization is to be considered, its capacity is to be defined in terms of a critical lane volume concept (1). This will be elaborated on later.

The second set of constraints is developed to define the distribution of the movements through the interchange. This set of constraints ensures that the distribution of traffic among the various movements entering the interchange will be the appropriate distribution. If V_1 is 20 percent of the total volume entering the interchange and V_2 is 10 percent of the total, V_1 will always be twice V_2, and the following relationship is true:

$$V_1 - 2V_2 = 0$$

All possible movements can be interrelated in this manner. Therefore, it is quite obvious that accurate count data or accurate estimates of future distribution must be available to develop these constraints.

Period of Analysis

The analysis can be conducted on a peak-hour basis or a 24-hour basis. However, it is recommended that a peak-hour analysis be used for the following two reasons:

1. Peak-hour capacity must be adequate for the interchange to operate efficiently during these periods; and
2. The afternoon peak-period traffic patterns generally differ from the morning peak patterns.

FORMULATION OF THE MODEL FOR A DIAMOND INTERCHANGE CONFIGURATION

The following is the formulation of the linear programming model for a conventional diamond interchange configuration, with signalization (Fig. 1).

Development of the Objective Function

For this particular interchange all 12 possible movements are permitted. These are defined as shown in Figure 1. Therefore, the objective function is $V_1 + V_2 + V_3 + V_4 + V_5 + V_6 + V_7 + V_8 + V_9 + V_{10} + V_{11} + V_{12}$, which is to be maximized.

Development of Constraint Equation

Figure 1 shows the identification of the physical constraints of the interchange. A capacity constraint is placed on each major geometric element. The next step is to develop the physical constraint equations shown later. Basically each constraint states that, for a particular geometric element of the interchange, the volume of the movements using the element is less than or equal to its capacity.

$$\begin{aligned}
V_1 + V_9 &\le C_1 \\
V_{10} + V_{12} &\le C_2 \\
V_3 + V_7 &\le C_3 \\
V_4 + V_6 &\le C_4 \\
V_3 + V_7 + V_{11} &\le C_5 \\
V_1 + V_5 + V_9 &\le C_6 \\
V_2 + V_6 + V_{10} &\le C_7 \\
V_4 + C_8 + V_{12} &\le C_8 \\
\text{critical lane volumes} &\le C_9 \\
V_7 + V_8 + V_9 &\le C_{10} \\
V_{10} + V_{11} + V_{12} &\le C_{11} \\
V_1 + V_2 + V_3 &\le C_{12} \\
V_4 + V_5 + V_6 &\le C_{13}
\end{aligned}$$

The capacity C_9 is the capacity of the sum of the critical lane volumes (1) passing through the two traffic signals. To illustrate this procedure, consider the following example. Movements V_1 and V_2 enter the signalization from the west and are equally distributed over two approach lanes (such might be the case when V_1 and V_2 are relatively low). Therefore, half of V_1 and half of V_2 make up the critical lane volume from the west (Fig. 2). From the two-lane east approach, V_7 and V_8 pass through the signalization in separate lanes, but, because V_7 is greater than V_8 (for the example considered), V_7 is the critical lane volume for the east approach. The one-lane north approach to the signalization is used only by movement V_{10}; therefore, V_{10} is a critical lane volume on this approach. The only movement that enters the signalization by the one-lane south approach is V_4, and it must also be considered a critical lane volume. For this particular example the constraint equation for the signalization (C_9) is

$$0.5V_1 + 0.5V_2 + V_4 + V_{10} + V_7 \leq C_9$$

If two lanes were provided for the V_7 movement, that is, a double left turn for the east to south movement, the critical lane volume on the east approach would be $0.5V_7$ if $0.5V_7$ were greater than V_8. However, if V_8 were greater than $0.5V_7$, then V_8 would be the critical lane volume for the east approach.

It can be seen at this point that the physical characteristics of the entire interchange are modeled by the selection of the 13 capacity values (C_1 through C_{13}) and by the signalization capacity constraint coefficients.

Data given in Table 1 illustrate the development of the interrelationship constraints. From these data the 11 interrelationship constraints are developed and are presented as follows:

$$\begin{array}{ll}
13.7V_1 - V_2 = C_{14} = 0 & 7.5V_1 - V_8 = C_{20} = 0 \\
15.3V_1 - V_3 = C_{15} = 0 & V_1 - V_9 = C_{21} = 0 \\
4.3V_1 - V_4 = C_{16} = 0 & 3.0V_1 - V_{10} = C_{22} = 0 \\
48.4V_1 - V_5 = C_{17} = 0 & 32.4V_1 - V_{11} = C_{23} = 0 \\
10.8V_1 - V_6 = C_{18} = 0 & 3.0V_1 - V_{12} = C_{24} = 0 \\
26.5V_1 - V_7 = C_{19} = 0 &
\end{array}$$

As discussed previously, either a peak-hour or a 24-hour analysis may be conducted. If separate analyses of the morning and afternoon peak periods are conducted, two sets of interrelationship constraints must be developed, one set defining the morning peak-period vehicular distribution and the other defining the afternoon peak-period distribution, and two linear programming models are to be solved independently. One model will have the morning peak-period vehicular distribution, and the second model will have the afternoon distribution.

Assignment of Capacity Values

Thus, with the objective function defined and the constraint equations developed, only assignment of specific capacity values to the physical constraint equations remains before the linear programming problem is solved. The hourly capacity values are a function of the level of service at which analysis is to be conducted. For level of service E, as defined by the 1965 Highway Capacity Manual (2), the following hourly capacities are applicable:

Facility	Design	Capacity
Arterial street (minor)	Two lanes, one direction	3,000 vphg
	Three lanes, one direction	4,500 vphg
Freeway	Two lanes, one direction	4,000 vph
	Three lanes, one direction	6,000 vph
Ramp	One lane	1,500 vph
	Two lanes	3,000 vph

The capacities given for arterial streets (measured in vehicles per hour of green time) cannot be obtained if interference from nearby signalized intersections exists. If such is the case, some provision must be made to reflect these conditions.

Establishing an hourly capacity of the signalization is done with respect to the critical lane volume concept (1).

Figure 1. Identification of movement and physical constraints for a conventional diamond interchange.

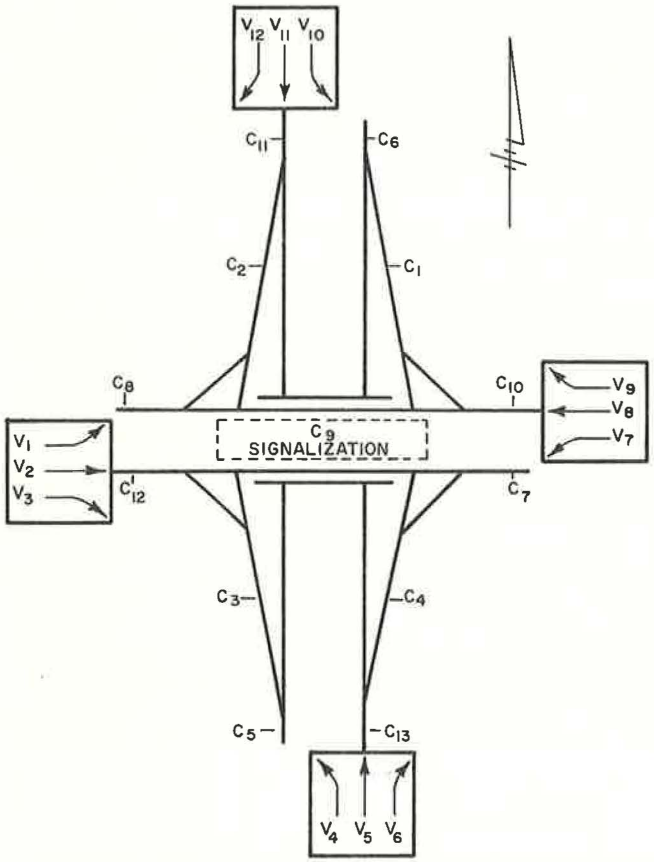

Figure 2. Lane use for determining critical lane volumes.

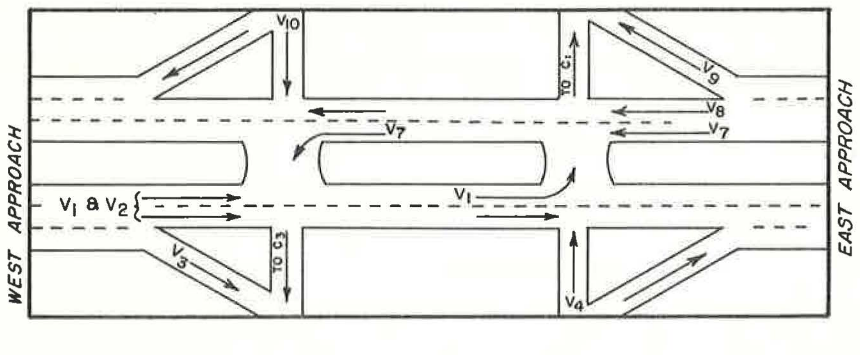

Table 1. Peak-hour distribution of entering traffic.

Movement	Percentage of Total Volume Entering Interchange	Movement	Percentage of Total Volume Entering Interchange
V_1	0.6	V_7	15.9
V_2	8.2	V_8	4.5
V_3	9.2	V_9	0.6
V_4	2.6	V_{10}	1.8
V_5	29.0	V_{11}	19.4
V_6	6.4	V_{12}	1.8

Solution of the Model

The linear programming model for the interchange is now complete, and the next step is to solve the problem. The simplex method is a mathematical procedure for solving linear programming problems. It can best be described as a technique of matrix algebra used to obtain the optimum values for the objective function. However, because of the widespread application of linear programming, numerous computer programs are available for solving such problems. IBM's Mathematical Programming System (MPS) was used for this study.

INTERPRETATION OF RESULTS

The purpose of the physical and interrelationship constraints is to define the area of possible solutions that will maximize the objective function. At least one of the physical constraints will be the critical constraint, and in the optimal solution that constraint becomes an equality. It is possible for two or three physical constraints to become critical simultaneously. The solution will indicate the value of the objective function, the values of each of the vehicular movements, the critical physical constraint or constraints, and the unused capacity of the physical constraints that are not critical.

The value of the objective function is interpreted as the maximum total volume that may be accommodated by the interchange configuration before congestion begins to develop. The critical element, that element on which the volume equals the element's capacity, is also identified. This is interpreted as the element at which congestion will first develop and is, therefore, the segment that limits the overall capacity of the interchange. The critical element could be any of the 13 elements shown in Figure 1. By identifying the critical element or elements, the designer can direct his attention to the needed areas of improvement and can modify these elements to increase their capacities. Thus, the linear programming model can aid the designer in developing new interchange configurations that provide higher capacities than the one currently under analysis.

CONCLUSIONS

The linear programming model provides the highway designer with an effective mathematical tool for the evaluation of the operational characteristics of an interchange subject to basic configuration, physical features, and traffic patterns.

With this ability, the designer can more effectively consider the problems of vehicular interactions and peak-period congestion within the framework of an interchange design sequence.

Wattleworth and Ingram (3) cover a case study in which the linear programming model was applied.

ACKNOWLEDGMENT

The authors wish to acknowledge the active cooperation and support of the Florida Department of Transportation, which sponsored this research.

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CAPACITY AND LEVEL OF SERVICE CONDITIONS ON DANISH TWO-LANE HIGHWAYS

Jens Rørbech, Technical University of Denmark

In 1969 recordings were carried out on four sections of rural highways in the eastern part of Jutland, Denmark. The purpose of these recordings was to provide Danish data for a decision on whether the particulars provided in the Highway Capacity Manual for two-lane highways were also valid and could be applied to Danish conditions. The recordings were organized as a sample survey of the correlation between operating speed and traffic volume (with different percentages of heavy commercial vehicles) for road sections with different free operating speeds and different gradient and visibility conditions. At each of the four locations, stationary recordings as well as tests by moving test vehicles were carried out. The stationary recordings were carried out by means of a magnetic tape recorder. The test runs were carried out by means of a tachograph and were designed for the recording of overall journey times over distances varying from 3.4 to 6.0 km. All observations show that cars in Denmark, all other factors being equal, are driven faster than is indicated by the Highway Capacity Manual. Based on the Danish observations, a set of curves has been worked out depicting the correlation between operating speed and volume-capacity ratio for free operating speeds ranging from 50 to 100 km/hr and for different sight conditions. Special problems are at last subject to a closer examination, such as the importance of heavy vehicles and measurements of operating speed.

•DURING August 19-24, 1969, systematic recordings were carried out on four sections of rural highways in the eastern part of Jutland, Denmark. The purpose of these recordings was to provide data for a decision on whether the particulars provided in the Highway Capacity Manual (1) for two-lane highways were also valid for Denmark and could be applied under Danish conditions.

In Denmark, new directives for the planning of highways have been worked out for the last few years. For a country like Denmark, it is of decisive importance to determine to what degree recommendations based on foreign data are applicable.

The recordings were organized as a sample survey of the correlation between operating speed and traffic volume (with different percentages of heavy commercial vehicles) for road sections with different free operating speeds and different gradients and visibility conditions. The four locations are given in Table 1.

The width of the pavement at all four locations is 8.0 m (two lanes each 4.0 m including marginal strips). Danish car lengths vary from 3.0 to 5.0 m, the typical length being about 4.0 m.

The results of the survey have been published in a report (2). Apart from the survey results, the report provides some guidance for the calculation of capacity and level of service on Danish two-lane highways. For this purpose, the definitions and notions quoted in the Highway Capacity Manual have been adopted without change; however, to the extent to which it was possible to use the survey for an adjustment to Danish conditions of the values of the input parameters, this was done. The most important results of the survey are discussed below.

THE SURVEY

At each of the four locations, stationary recordings, as well as tests by moving test vehicles, were carried out.

The stationary recordings were carried out by means of the magnetic tape recorder shown in Figure 1. With the aid of four contact cables, the passage times of all motor cars were recorded separately on each lane. On the Vejle to Kolding road, where the recordings were taken on a 33 per mil gradient, and on the Kolding to Middelfart road, where the gradient was 55 per mil, traffic was recorded at two points about 275 m apart.

The magnetic tape recordings were processed on an IBM 7094 computer at the Technical University.

For each passage of a motor car, the following data were (inter alia) derived: arrival time, headway time, headway space, speed (travel speed over the cable distance of 4.0 m), acceleration, and wheelbase and axle combination.

Mean values were calculated for 5-min intervals (inter alia) for traffic volumes, mean speed, traffic density, and percentage of heavy commercial vehicles.

The results of the stationary recordings obtained at the four locations are shown in Figures 2, 3, 4, and 5 in the form of quarterly figures for both directions together.

The diagrams show correlated values of the operating speed as a function of the volume-capacity ratio. Capacity is defined as

$$c = 2{,}000 \times T_c$$

where T_c is a reduction factor (measured in cars per hour) that decreases with the incidence of heavy vehicles (e.g., trucks) and is calculated from the truck equivalency values given in Table 2. The operating speed is determined from the observed mean speeds by adding a correction value as described later.

The test runs were designed for the recording of overall journey times over distances varying from 3.4 to 6.0 km.

Installed in the test vehicle was a tachograph that, during the run, records the speed of the vehicle as a function of time. During each trip, the passages not only over the fixed starting and finishing points but also over the measuring cables were recorded. A special pen was used for recording driving in queues, the number of overtakings, and the number of cars overtaken. The speed of the test vehicle was closely adapted to the traffic flow, in accordance with the definition of operating speed.

Because the test runs were carried out at the same time as the stationary recordings, it has been possible to compare the results. The individual test run results show, as might be expected, a much greater scatter than the stationary recordings. Allowing for the greater scatter, there is however good agreement between both sets of results.

The observations at the four locations were carried out at different times of the day and on different days of the week so as to obtain data on the distribution of traffic flows and truck percentages. Altogether, the survey comprised about 12 quarterly observations and about 30 test runs at each location.

DIRECTIVES FOR THE CALCULATION OF CAPACITY AND LEVEL OF SERVICE

Correlation Between Operating Speed and Volume-Capacity Ratio

If the Danish speed recordings are compared with those quoted in the Highway Capacity Manual (the curves from the Manual corresponding to the geometrical conditions at the survey locations are shown in Figures 2, 3, 4, and 5) it will be seen that, for a given traffic flow, traffic in Denmark moves faster than is indicated by the American data. This difference is equally clear from the results of all the four survey locations and is of an order of magnitude that cannot be simply ignored.

With this observation in mind, curves that take this difference into account have been plotted and are governed by the following conditions.

Table 1. Study locations and characteristics.

Location	Free Operating Speed (km/hr)	Sight Distance (percent)	Terrain
Highway A10 between Christiansfeld and Kolding	100	75 to 100	Level road
Highway A10 between Vejle and Kolding	100	75 to 100	Moderate gradients
Highway A10 between Hasselager and Hørning	90	50 to 75	Moderate gradients
Highway A1 between Kolding and Middelfart	60	0 to 25	Heavy gradients

Figure 1.

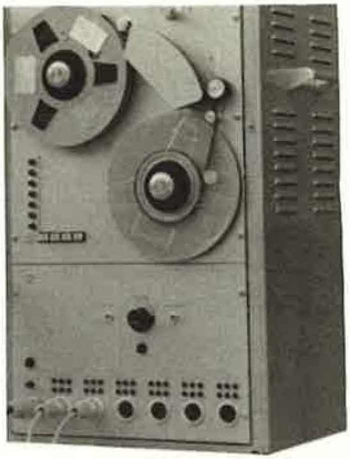
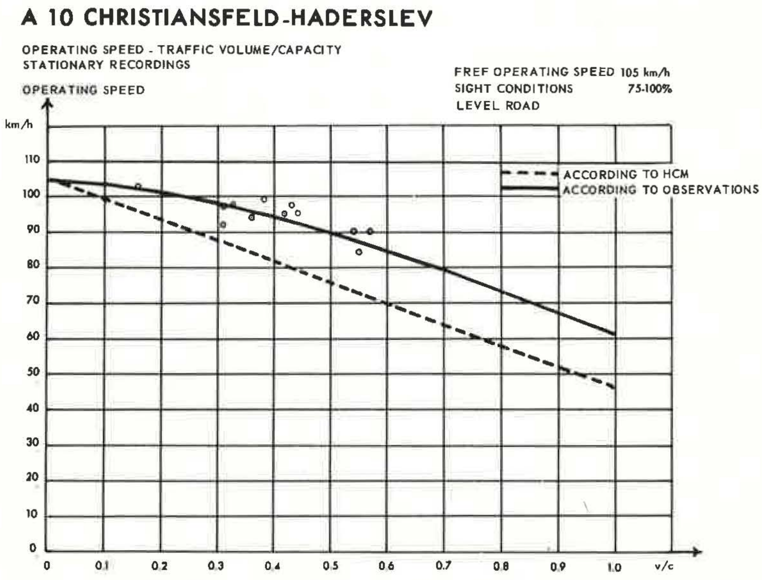


Figure 2.



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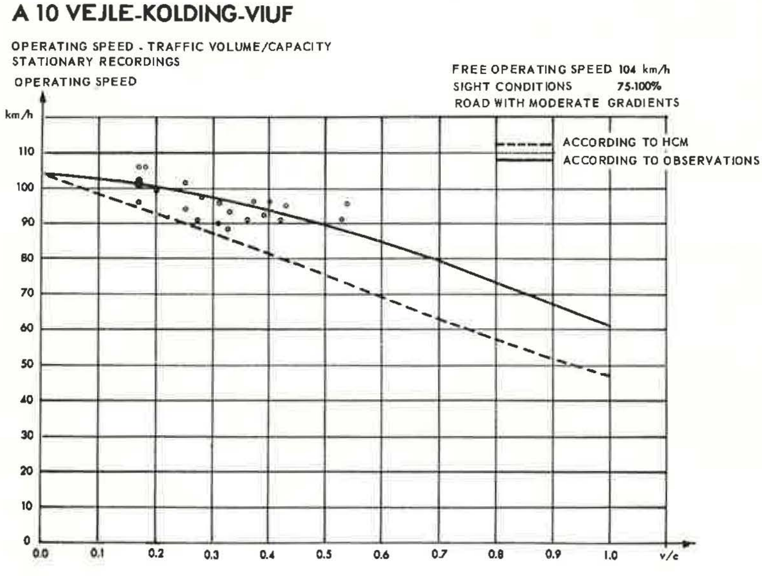

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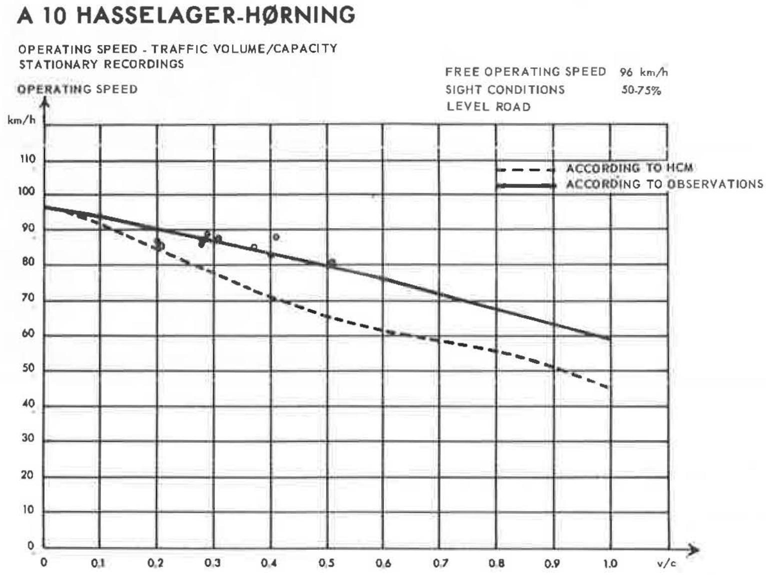

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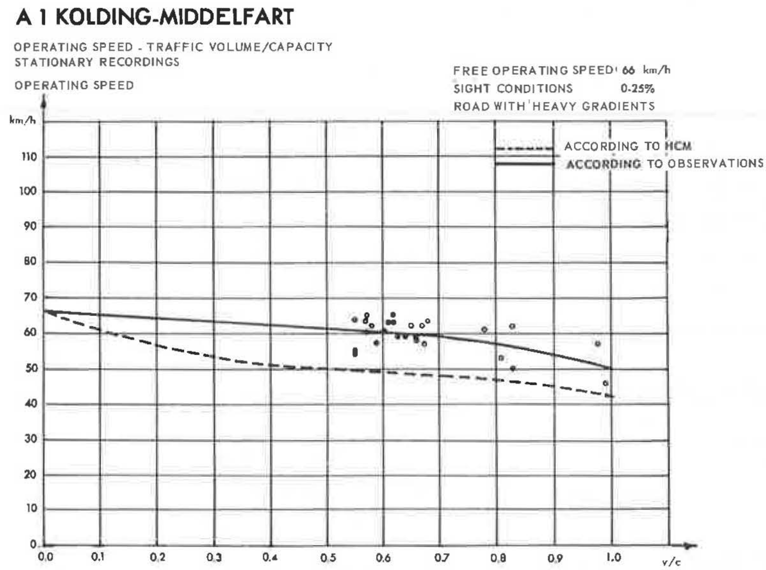


Table 2. Truck equivalency values for study locations.

Highway	Terrain	Truck Equivalency Value
A10 Christiansfeld to Haderslev	Level road	2
A10 Vejle to Kolding	33 per mil gradient at about 500 m	5
A10 Hasselager to Hørning	20 per mil gradient at about 125 m	2
A1 Kolding to Middelfart	55 per mil gradient at about 400 m	8

1. The curves are intended to form the basis of a set of curves that cover all the free operating speeds and sight conditions encountered in practice and will therefore appear in the cigar-like shape known from the Highway Capacity Manual.
2. The extreme left point of the curve, representing the free operating speed over the section of road, can be separately determined as the 85th percentile of the cumulative frequency curve of the speed of unimpeded passenger cars.
3. The curve for highway A10 between Christiansfeld and Haderslev should be identical with that for highway A10 between Vejle and Kolding where the traffic volumes are fairly low because there is little difference in the free operating speeds (105 and 104 km/hr respectively).
4. For traffic flows close to the capacity limit, the curves should be governed by the results obtained on the Kolding to Middelfart road where traffic volumes close to the capacity limit have been observed.

It was possible to use the curves plotted in Figures 2, 3, 4, and 5 as background information for a set of curves depicting the correlation between operating speed and volume-capacity ratio for free operating speeds ranging from 50 to 100 km/hr and for sight distance percentages of 0 to 25, 25 to 50, 50 to 75, and 75 to 100 percent (Figs. 6 through 11).

The subdivision of the diagrams showing the operating speed as a function of the volume-capacity ratio into the service levels A, B, C, D, and E depends on the observed correlations between operating speed and volume-capacity ratio. In the Highway Capacity Manual, the subdivision is based on the assumption that the intersecting points between the horizontal and vertical limits are located exactly on the curve for a free operating speed of 70 mph with 100 percent sight distance (Fig. 12a). Because the Danish operating speed differs from the American one, a proposal has been worked out for varying the level of service classification on Danish highways (Fig. 12b).

Influence of Heavy Vehicles

The capacity definition given in the Manual contains the factor T_c, which reduces the capacity if the traffic includes heavy vehicles. Because the presence of heavy vehicles has an effect on the capacity and because the traffic load itself is given as the total number of passing cars (without converting the number of trucks into passenger car units), the method of quoting the traffic loads in terms of passenger car units has been formally abandoned. The load is now quoted purely as the volume-capacity ratio during the interval.

In practice, however, this new method is not different from the one based on quoting the traffic load in passenger car units, inasmuch as the reduction factor T_c is based on the determination of the truck equivalency coefficient. All that are vital to the calculations are, therefore, to determine whether it is altogether reasonable to convert heavy vehicles into a corresponding number of passenger cars and, if so, to fix the relevant truck equivalency coefficients.

Although the Danish surveys were not specially designed to provide data concerning these issues, both questions have been investigated with a view to clarifying the many problems associated with the application of the truck equivalency coefficients.

The attempt has been made to use the individual speed recordings in such a way that observations of low truck percentages were compared with observations of high truck percentages, partly to ascertain the difference in the structure of the results and partly to find the most relevant correlation factors (truck equivalency coefficients). It was found that the individual speed recordings had such a great scatter that it was not possible to indicate any clear correlations, and the survey result thus gives rise to doubts of whether it is appropriate to convert heavy vehicles into a corresponding number of passenger cars.

It is however possible, based on the strength of the recordings from highway A1 between Kolding and Middelfart, to check whether the American truck equivalency coefficients could reasonably be applied to Danish highways. For this section of road, the equivalency coefficient amounts to $E_T = 8$, which is in agreement with the Manual.

Figure 6.

CORRELATION BETWEEN OPERATING SPEED AND v/c

FREE OPERATING SPEED 50 KM.P.H.

BASED ON DANISH OBSERVATIONS

OPERATING SPEED

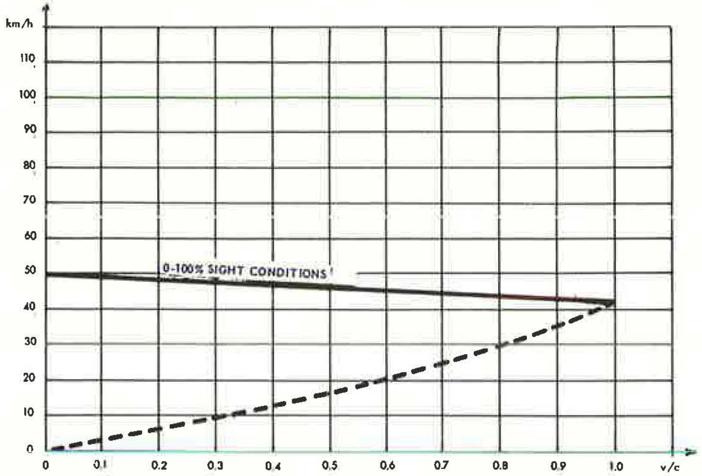


Figure 7.

CORRELATION BETWEEN OPERATING SPEED AND V/C

FREE OPERATING SPEED 60 KM.P.H.

BASED ON DANISH OBSERVATIONS

OPERATING SPEED

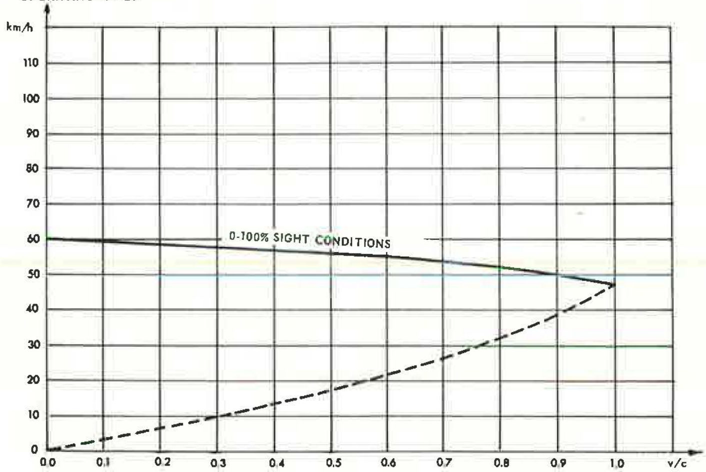


Figure 8.

CORRELATION BETWEEN OPERATING SPEED AND V/C

FREE OPERATING SPEED 70 KM.P.H.

BASED ON DANISH OBSERVATIONS

OPERATING SPEED

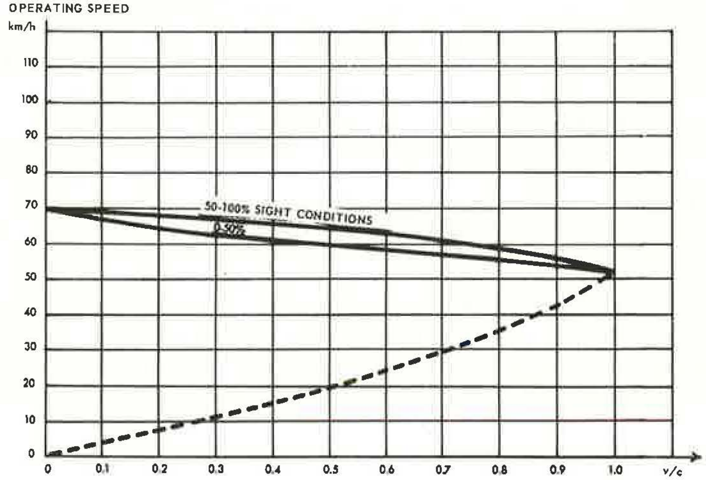


Figure 9.

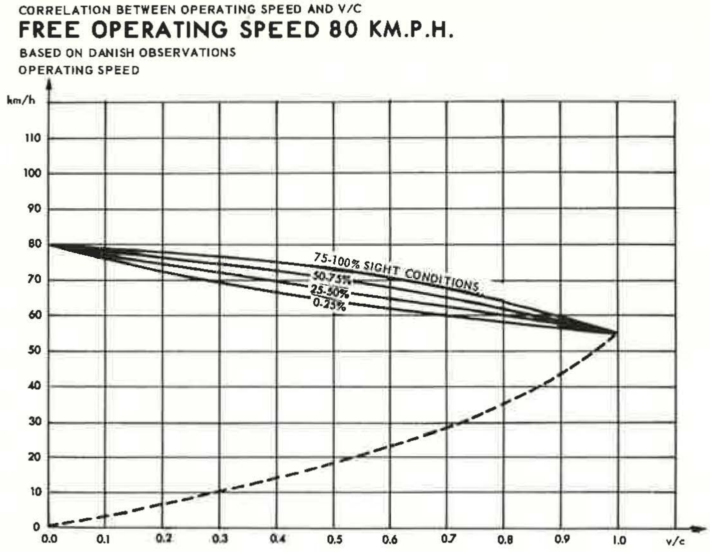


Figure 10.

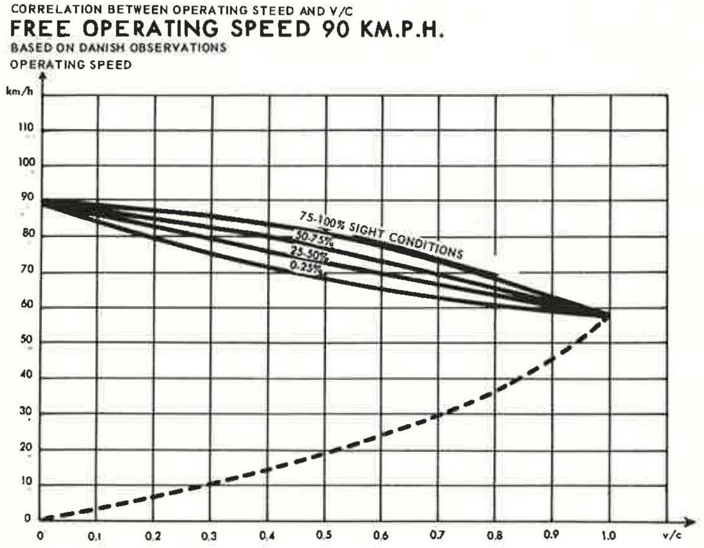


Figure 11.

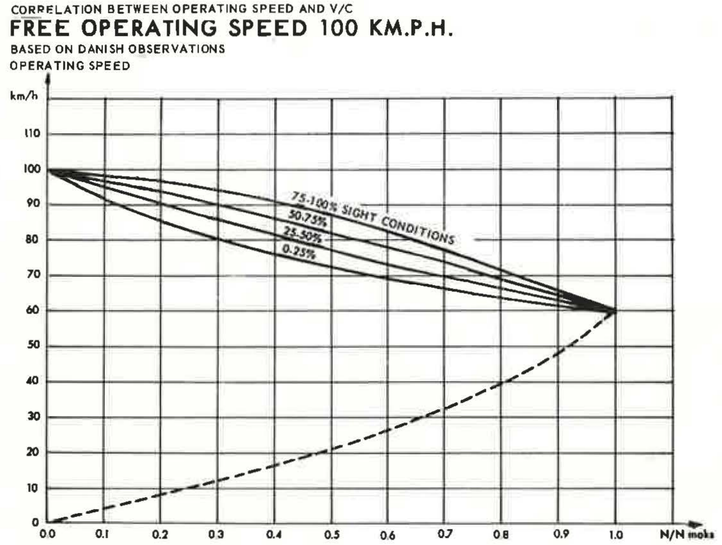
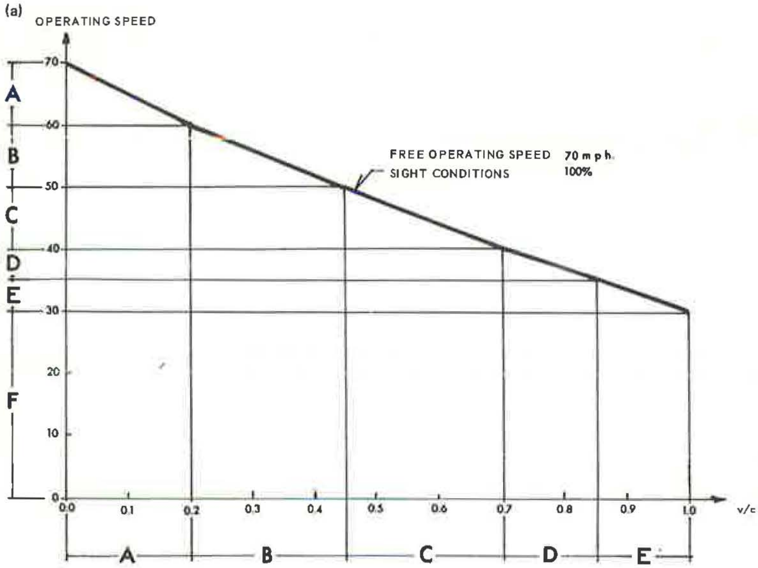


Figure 12.

(a)



(b)

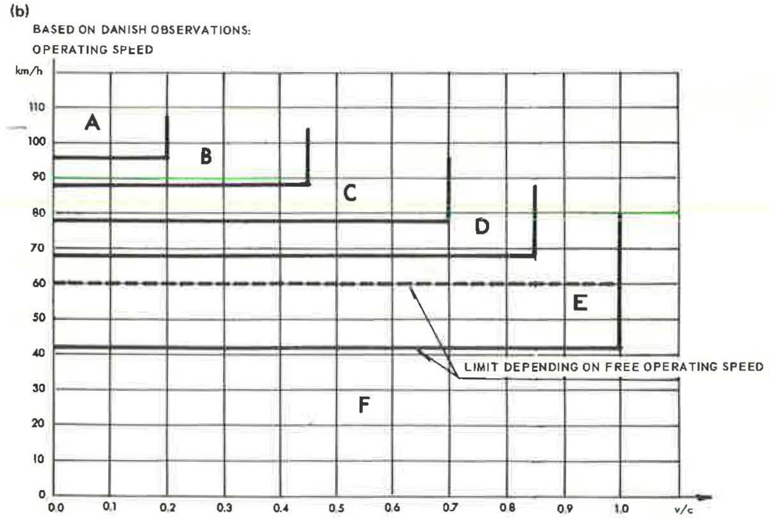
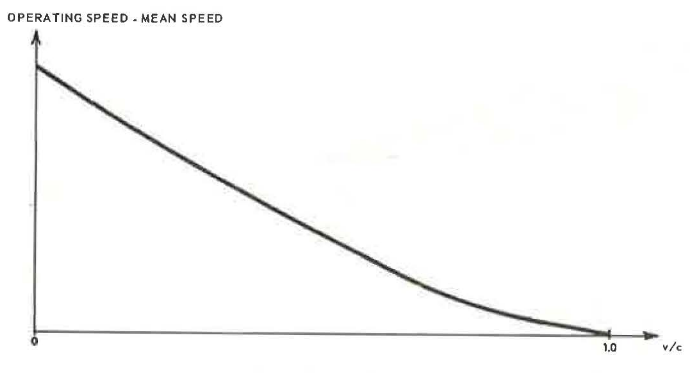


Figure 13.



The application of this truck equivalency coefficient means that the traffic load on this road is often around the capacity limit (Fig. 6). If one stands near the road during that period, there is no doubt that the load is close to the capacity limit. This is also corroborated by data obtained from a permanent counting station near the survey point, which shows frequent maximum loads around the capacity based on $E_T = 8$ but no loads much in excess of that figure.

On the two-lane highway A2 between Holbaek and Roskilde on the island of Zealand, which can be classified as a road with moderate gradients (Table 1), similar calculations yield an E_T value of 4 to 5, and the E_T value for roads with moderate gradients is in fact proposed to be taken as $E_T = 4$. The proposal for level roads is $E_T = 2$.

Measurement of Operating Speeds

In fixing the level of service for a recorded traffic situation, the Manual assumes a knowledge of the operating speed but does not indicate how an observed mean speed can be converted into an operating speed. In the investigation, the following method has been adopted.

It is well known how the speed distribution changes with different traffic volumes; with increasing traffic volume, both the mean speed and its standard deviation are reduced.

The operating speed is the highest speed that a responsible driver will choose in given situations; it has therefore been fixed that the operating speed corresponds to the 85th percentile speed distribution, i.e., 15 percent of the vehicles are driven faster than the operating speed. The 85th percentile figure has been fixed arbitrarily; it is the same figure that has been used in fixing the speed limits. This definition is also supported by a number of laboratories abroad (4, 5).

In this way, the operating speed can be fixed when the momentary speeds of individual vehicles and therefore the speed distribution of the vehicles are known. This would, however, call for a not inconsiderable effort in determining the 85th percentile figure, and a somewhat easier method can also be used: If the operating speed is defined by the 85th percentile, the operating speed will show a greater decrease with increasing traffic volume than the mean speed; i.e., the difference between operating speed and mean speed has the form shown in Figure 13. By comparing the correlations between mean speed and volume-capacity ratio and between operating speed and volume-capacity ratio given in the Manual, one arrives at an almost straight-lined curve. The point of intersection of this curve with the ordinate axis depends on the free operating speed so that the difference increases with the free operating speed (Fig. 14).

A set of curves corresponding to Figure 13 has been prepared for free operating speeds ranging from 50 to 100 km/hr (Fig. 15), based on speed distributions obtained from the four survey locations. Each distribution is plotted for 100 unimpeded passenger cars, and the difference between the 85th percentile and the 50th percentile is read off. Figure 16 shows the result from the highway A10 between Hasselager and Hørning.

CONCLUSION

The Danish observations have not permitted a check on (a) the basic factor of 2,000 cars per hour in the capacity equation or (b) the effect of the lane width on capacity. As regards the latter, the directives have been adapted to the widths of Danish roads on the basis of the American data alone.

The main finding of the survey is that we in Denmark, other things being equal, drive at higher speeds than those indicated in the Highway Capacity Manual. Similar differences have also been found in other European countries. The question is whether this difference is due to a difference in driving habits between Americans and Europeans or to the fact that the data on which the Highway Capacity Manual is based are now some years old.

For a volume-capacity ratio of 0.5, the difference between the Danish and American observations amounts to about 10 to 15 km/hr. If it is assumed that the basic material of the Highway Capacity Manual is by now about 10 years old, the difference agrees

Figure 14.

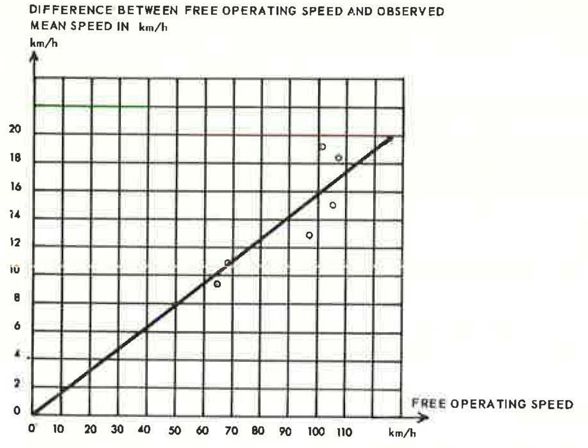


Figure 15.

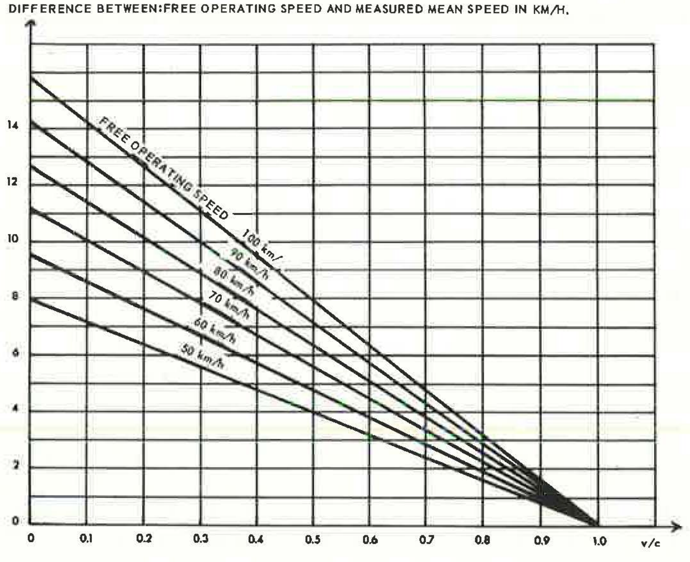
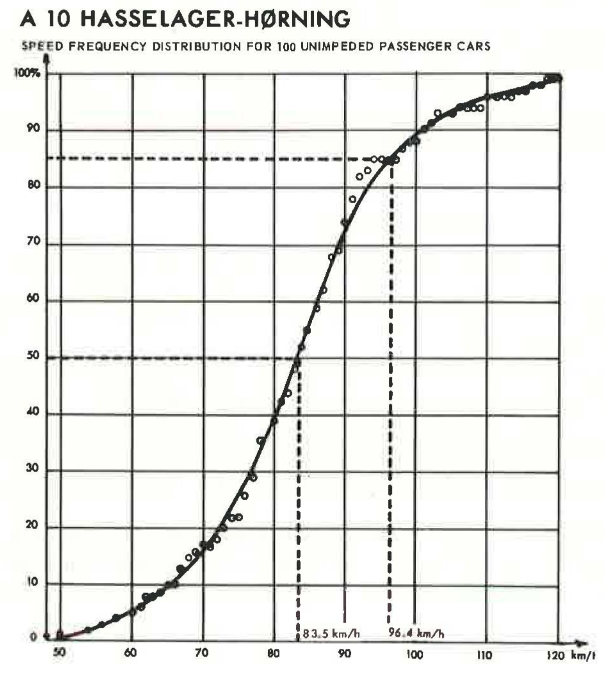


Figure 16.



quite well with the general speed increase, experienced in many places, of approximately 1 to 2 km/hr per annum. The Highway Capacity Manual itself refers to observed data showing an increase in the measured mean speeds from 45.2 mph in 1946 to 55.6 mph in 1964, corresponding to 0.9 km/hr per annum. But these speed observations had been carried out during periods of light traffic and thus merely reflect a general desire of being able to increase the free operating speeds. Carrying out the comparisons on Figures 2, 3, 4, and 5 in such a way that the extreme left point of the curve ($v/c = 0$; i.e., the speed corresponds to the free operating speed) is the same for the Danish and American observations should allow for this raised demand for a higher free operating speed.

It thus remains to state that the speed chosen by Danish motorists is less sensitive to increases in traffic volume. Whether this is a specifically Danish or European phenomenon cannot be decided at present, and it is possible that a similar trend could now also be discerned in America. On the other hand, it is not improbable that, because of the rapid increase in the Danish car ownership rates, we have not, to the same degree as in America, learned to drive properly in heavy traffic.

One must be aware that the motorists' speed adaptation to the momentary traffic volume is first and foremost a question of road safety. In fixing the different levels of service, one ought to take into account road safety considerations. As Figure 12 shows, the classification of the levels of service is based on a recording in accordance with the best conditions that can be obtained, without providing any guarantee that a traffic performance corresponding to the curve in Figure 12 does, in fact, take reasonable account of all the factors, including road safety. One should therefore try to determine how the adaptation of the speed takes place so that one may, possibly through the introduction of new explanatory factors, create a better basis for fixing those speeds that would be reasonable to use.

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