Examination of the Speed-Flow Relationship at the Caldecott Tunnel

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Recently, a new procedure for analyzing multilane highways was proposed, together with a set of speed-flow curves that were rather different from those in the *Highway Capacity Manual* (HCM). Consideration is given to whether a freeway basic section treated as a multilane divided highway section with no access points can be analyzed using this procedure. The speed and flow characteristics of a freeway section (California State Highway 24) at the Caldecott Tunnel are examined and compared with the speed-flow curves in the HCM for freeways and those in the new procedure for multilane highways. The problems associated with traffic data gathering, reduction, and analysis are also reviewed. It was found that the new procedure reflects the speed-flow characteristics at the Caldecott Tunnel better than the HCM procedure for freeways.

The relationship among speed, density, and flow of traffic on a highway has been the subject of many years of intensive research, resulting in numerous publications on theoretical models and empirical investigations of the traffic characteristics of highway sections. The first traffic flow model postulated by Greenshields assumed a linear relationship between speed and density, giving a parabolic speed-flow function. This shape has influenced the understanding of the relationship between speed and flow on highways for many years.

Traditional speed-flow curves as represented by those in the different versions of the *Highway Capacity Manual* (HCM) (*1,2*) are taken to be smooth, that is, differentiable and continuous functions over the entire range of density values. Although the 1985 HCM has noted some studies (*3,4*) indicating that speed-flow relationships may be better described by nondifferentiable and perhaps discontinuous speed-flow functions, the results of these studies have not been incorporated in the HCM calibrated curves. Adopting a continuously differentiable speed-flow function necessitates the bending of the speed-flow curve and a precipitous speed drop near capacity. This condition is clearly seen in the HCM speed-flow curves, in which the optimum speed is always at about 30 mph for freeways and multilane highways, regardless of the geometrics.

Recently, a set of speed-flow curves for multilane highways has been proposed (*5*) as a revision to Chapter 7 of the HCM. This new method is referred to here as the JHK method. Figure 1 shows the set of speed-flow curves proposed. Several interesting differences become apparent when these curves are compared with those currently used in the HCM for multilane highways and freeways, as shown in Figure 2. These differences include the following:

- The mean passenger car speed, instead of the average travel speed, is used as the speed variable in the speed-flow relationship.
- Only the upper branch of the speed-flow curves dealing with uncongested flows is presented in the JHK method. Comparing this method with the HCM curves indicates that the former predicts a drastically smaller change in speed with increased flows. The mean passenger car speed remains constant for flows up to 1,400 passenger cars per hour per lane (pcphpl) and subsequently drops gradually and nonlinearly as capacity is reached. The total drop in speed in the upper branch is always 5 mph for all curves. In contrast, the HCM curves predict a gradual drop in speed as flow increases, followed by a precipitous drop in speed near capacity. Clearly, the shape of the JHK curves implies that the speed-flow function is nondifferentiable at capacity.
- In the set of JHK curves, capacity is at 2,200 pcphpl under ideal conditions and is independent of the design speed of the highway. Indeed, capacity is constant regardless of the effects of geometric inadequacies, such as reduced lane width and lateral clearance. The HCM curves suggest that the capacity of multilane highways and freeways under ideal conditions is 2,000 pcphpl except when the design speed is 50 mph, in which case the capacity is 1,900 pcphpl.

Besides these differences in the speed-flow curves, there are several other differences related to the two methods of the operational analysis of multilane highways:

- Development environment, which has been classified as rural and suburban in the HCM procedure for multilane highways, is replaced by a more precise measure of access density in terms of the number of access points per mile in the JHK method.
- As in the HCM procedure, multilane highways are classified as divided or undivided in the JHK procedure. However, highways with continuous left-turn lanes separating opposing flows are considered specifically as divided highways in the JHK method instead of being somewhat in between divided and undivided highways in the HCM procedure.
- In the JHK method, lane width, lateral clearance, type of median, and number of access points affect not the capacity but the operating speed of the highway. In the HCM method, reduction in lane width, insufficient lateral clearance, or less-than-ideal median type and development environment result in a reduction in highway capacity. Hence, adjustment because of nonideal geometrics (not including terrain type) causes

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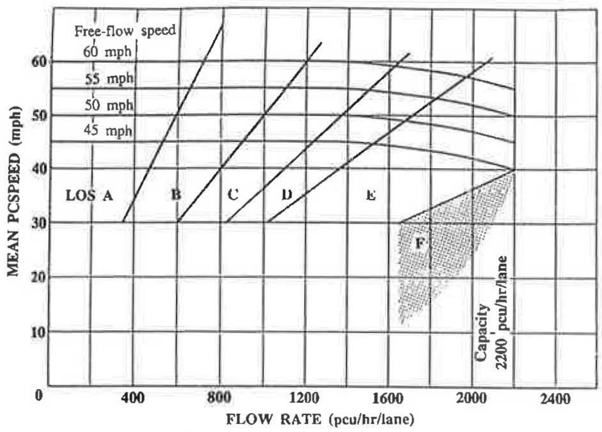


FIGURE 1 JHK speed-flow curves for multilane highways (5).

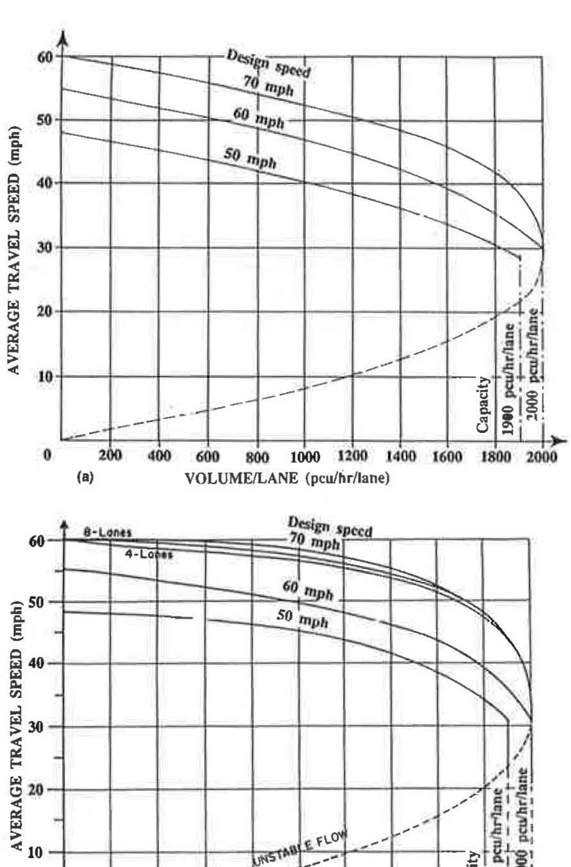


FIGURE 2 HCM speed-flow curves: (a) multilane highways; (b) freeways (2).

a vertical translation of the ideal speed-flow curve in the JHK method but a change in the horizontal scale of the speed-flow curve in the HCM method.

- The JHK method uses a new set of values for heavy-vehicle factors. In general, trucks and recreational vehicles have lower equivalent factors than those in the HCM. Furthermore, no distinction is made between trucks and buses for the purpose of passenger car unit (pcu) conversion.
- Adjustment for noncommuter traffic in the JHK method is considered unnecessary and therefore is not included.
- The density values that divide the various levels of service are also revised, with the exception of levels of service (LOS) A and B. Furthermore, because the capacity may occur at any speed in the new method, the maximum density value for LOS E is not fixed as it is in the HCM.

If the JHK method, which is meant for multilane highways, is accepted in the HCM, some disparity may exist between the design of multilane highway sections following the JHK procedure and the design of freeway sections following the procedure of Chapter 3 in the HCM. For example, a multilane divided highway with less than one access point per mile and designed for 70 mph under ideal conditions is expected to yield a higher capacity than a freeway section under ideal conditions with the same design speed. Consequently, if a freeway basic section were analyzed as a multilane divided highway section with no access points according to the new procedure, a better level of service and higher capacity could be obtained.

The speed-flow characteristics of a freeway section (California State Highway 24) near the Caldecott Tunnel are examined to see how well the observed conditions are predicted by the speed-flow curves in the two procedures: (a) by analyzing the freeway section according to the current HCM method for freeways and (b) by treating the freeway section as a multilane divided highway with no access points and analyzing it according to the JHK procedure.

First, recent studies of speed-flow relationships are reviewed, highlighting the problems encountered in making such studies. This review is presented in the next section, followed by a description of the Caldecott Tunnel site and the procedures of data collection and reduction. The results of the speed-flow analysis are then discussed, together with a comparison of the HCM and JHK curves.

RECENT STUDIES ON SPEED-FLOW RELATIONSHIPS

In recent years, several studies on the traffic characteristics of freeways (*6–10*) have challenged some of the traditional views of the relationship among speed, flow, and density. Except for Banks' work in San Diego (*9*), these studies relate Canadian conditions, with data taken mostly from Queen Elizabeth's Way near Toronto.

Several important issues have been raised in these studies, but basically these issues have to do with the way in which speed, flow, and density data are gathered, reduced, and analyzed. The common contention has been that serious misinterpretations of speed-flow relationships can result if unsuitable data reduction and analytical procedures are employed and if the influence of the study location on the nature of the data is disregarded. The problems associated with unsuitable analytical procedures, data gathering, and reduction are reviewed in this section.

The traditional method of analyzing speed, flow, and density data is to fit a function to the data. The problems related to fitting a curve to a set of speed-flow data have been pointed out by Hurdle and Datta (*4*) and Allen et al. (*6*). The form of the speed-flow curve, whether it is continuously differentiable, continuous but not continuously differentiable, or discontinuous, cannot be normally discerned from data alone. Yet the choice of the speed-flow function can have a significant impact on the interpretation of the speed-flow relationship. In order to avoid this problem, these researchers have departed from the usual way of fitting a curve to the data and have instead resorted to interpreting the speed-flow relationship by merely observing the scatter of data.

One consequence of not fitting a single function to the speed-flow data is that the speed-flow curve is examined in two portions: (a) the upper branch for the uncongested regime, in which the speed is seen to be relatively insensitive to flow, and (b) the lower branch for the congested regime. The main problem then lies in distinguishing the field data belonging to each portion—a complicated task when the flow is near capacity. Hall and Gunter (*7*) have even considered whether the speed-flow states can be between the two branches of the speed-flow curve by examining the issue of transition between the two regimes: flow breakdown and recovery.

Apart from the problems resulting from predetermining the speed-flow function, there are several other problems related to data gathering. All the researchers have reasoned that the location at which the data are gathered affects the nature of the data. Hall and Hall (*10*) have pointed out that the study location with respect to the bottleneck of the freeway is important—a point that was made long ago (*3*) but has not been seriously considered. A location upstream of a bottleneck may never experience flows near capacity because the flows are limited by the capacity of the bottleneck. A truncation in the speed-flow curve may result in this case. On the other hand, flows on a location downstream of a bottleneck may also be constrained by the bottleneck so that the true capacity of the location may not be recorded. Banks (*9*) reiterated this theory and noted that it may be difficult to find a location in which flows at capacity are observed. As pointed out by Hall and Hall (*10*), even though a location is characterized by the two portions of the speed-flow curve, in practice to obtain both of them for a particular location is difficult.

Persaud and Hurdle (*8*) have also indicated that speed data gathered in the acceleration zone downstream of a queue may be seriously misinterpreted. Because vehicles discharging from the queue accelerate over a finite distance, speeds observed in this zone may be influenced by the presence or absence of a queue. On this point, Hall and Hall (*10*) observed a reduction in speed in their site when there was an upstream queue, but Banks (*9*) observed no discernible change in speed in his site.

Serious problems may also arise because of data reduction procedures. For example, Allen et al. (*6*) have analyzed the data lane by lane, supposing that there are different speed-flow relationships between the different lanes, whereas Banks (*9*) has chosen to combine the data for all lanes, arguing that some portions of the speed-flow curve may not be observed otherwise. Persaud and Hurdle (*8*) also suggested that the averaging effect of taking data from different locations in a bottleneck may be the reason for the precipitous drop in speed near capacity observed in previous studies.

Related to the problems of data gathering and data reduction is the quality of the data gathered. Banks (*9*) has made use of data obtained from loop detectors installed on the freeways. These data are in the form of flow counts and occupancy. Speed is then determined indirectly, a computation that is found to be rather unreliable (*11*). On the other hand, Hurdle and his associates (*4,8*) have made use of time lapse photography to obtain a more direct, though not necessarily more accurate, measurement of speeds. The disadvantage they faced is that the sample size can be rather small. In order to increase the sample size for the study, the sampling time interval is shortened, to 2 min in their case. However, this change results in a greater variance in both speed and flow on the speed-flow plot, which may hamper a good evaluation of the change of speed with flow.

Another problem encountered by these researchers is the question of truck percentages. Persaud and Hurdle (*8*) selected a site at which trucks were not present, but, in the other studies, the proportion of trucks was estimated. All studies have assumed a truck equivalent factor of 2, but Banks (*9*) has avoided the problem by expressing flow in vehicle units rather than pcu.

In the light of these problems, it is extremely important that data be obtained and analyzed properly if a meaningful comparison is to be made between observed speed-flow conditions and predicted relationships.

DATA COLLECTION AND REDUCTION

The data used in this study are taken from the two westbound lanes that emerge from the northern bore of the Caldecott Tunnel through which California State Highway 24 passes. The Caldecott Tunnel has three bores: the southern bore is devoted to eastbound traffic, the northern bore to westbound traffic, and the center bore to reversible traffic flow. Typically, the center bore is open to westbound traffic in the morning and eastbound traffic in the afternoon.

The section under investigation is about 200 ft downstream of the tunnel exit. At this point, the two lanes are each 12 ft wide, and the lateral clearance is 6 ft on each side. The lanes are sloping, with a downgrade of 5.5 percent. An offramp from the freeway is located about 900 ft after the tunnel exit, and an onramp is located 600 ft further downstream. The flow on the off- and onramps, which primarily serve the control center above the tunnel, is so low that traffic on the main line is unaffected by the traffic on the ramps.

When the westbound flow exceeds the capacity of the tunnel, traffic backs up from the entrance of the 3,000-ft-long tunnel. Under normal conditions, traffic emerging from the tunnel is free flowing.

At the time of the study, two sets of detectors were installed about 200 ft downstream of the tunnel exit along the westbound lanes. These detectors were installed as part of a project funded by the California Department of Transportation (Caltrans) to study detector technologies for freeway surveillance and control. The first set of detectors are inductive loop detectors buried on the roadway on each of the two lanes. (These two lanes are referred to here as the shoulder and middle lane. There are other westbound lanes, but they use the other tunnel bore.) The second set of detectors are ultrasonic detectors suspended from a roadway overpass about 100 ft downstream of the loop detectors. Both sets of detectors are wired to a roadside junction box and connected to Fisher-Porters recorders and Caltrans 170 controllers located in the Caldecott Tunnel control room just above the study site. Every 1/60 sec, the 170 controllers would scan the various detector stations for the presence of a vehicle within the detection zones. Detector information is transmitted from the control center via a telephone line to a microcomputer for data logging at the Institute of Transportation Studies at the University of California in Berkeley.

The detector signals are transformed to pulses, from which durations of "on" and "off" times at each detector station can be determined. Irregular signals have been eliminated, and the possibility of detectors miscounting the vehicles has been studied by comparing the detector pulses with visual observation of the traffic recorded on a video recorder over a period of 2 hr. An average overcount of 0.4 percent was made by the loop detectors, whereas the ultrasonic detector overcounted on average by 2.0 percent. The computation of flow across the detectors is therefore based on the loop information.

Speed measurements are obtained not from occupancy data but by measuring the time of travel of a vehicle between the loop and ultrasonic detectors. Provided that the correct pair of pulses can be identified, it is possible to determine the speed of the vehicle by noting the time difference between the onset of the pulses. Matching the pulses is crucial to the correct evaluation of the speeds because any mismatch can lead to biased results. The criterion to identify the correct match is based on the computed speed of the vehicle associated with the pulses. If the pulses are wrongly matched, the computed speed will either be too high or too low. An arbitrary, though not unreasonable, upper limit of 100 mph and a lower limit of 20 mph are set to test the pulse matching. Matching proved to be rather insensitive to the values of the cut-off levels when they are near the two limits set. In most cases, the pulses matched sequentially. On the basis of observations made from the 2-hr video recording, the number of matched pulses accounts for about 99.5 percent of the cases. The computed speed values are certainly not affected by ignoring the unmatched cases.

In order to express flows in pcu, the type and proportion of heavy vehicles must be known. However, these characteristics cannot be obtained directly from the detector information. On-site observation shows that the number of recreational vehicles and buses is small compared with the number of trucks. It is therefore sufficient to consider just one class of heavy vehicles. Furthermore, it is possible to estimate the percentage of heavy vehicles by treating long vehicles as heavy ones. This calculation requires that the length of vehicles be known, which is obtained by noting the computed speed of individual vehicles and the corresponding recorded "on" times. Investigation into the distribution of "on" times of the loop and ultrasonic detectors shows that the loop detectors gave more reliable "on" times. The length of vehicles is therefore computed on the basis of the "on" times recorded by the loop detectors. Although the computed values of vehicle length are not precise, the estimated percentage of heavy vehicles is not significantly affected by this. According to a comparison of the detector pulses and the video recording, a threshold of 27 ft is found to distinguish long vehicles, including trucks, buses, and recreational vehicles, from passenger cars. Instead of applying a fixed pcu equivalent for the heavy vehicles (such as 2), the pcu equivalent factors used in this study are based on the recommended values given by the HCM and JHK methods.

A total of 131 hr of data was collected over several occasions from March 16 to April 18 of 1990. On the basis of detector information, the space-mean speeds (harmonic mean of individual speeds), vehicle counts, and proportion of heavy vehicles on each of the two lanes for various time intervals were obtained.

ANALYSIS OF SPEED-FLOW DATA

Speed-Flow Relationship

Figure 3 shows the speed-flow plot for the average lane using data grouped in 2-min, 15-min, and 1-hr sampling periods. For shorter periods, there is a greater scatter in the data in the measurements of speed and flow. However, the abundance of data points and the controlled measurements of speed have produced a tight band of the data in all three graphs (except where the flow is extremely low, resulting in a corresponding small number of speed measurements). Clearly, there is a well-defined relationship between speed and flow in the upper branch of the speed-flow curve.

Speed is insensitive to flow in the low-flow region for flows up to about 800 vehicles per hour (vph), beyond which the speed drops gradually but definitely until the maximum flow is observed. The graphs indicate that it is not correct to suggest that speed is insensitive to flow for the entire upper branch of the speed-flow curve. However, a precipitous drop in speed near capacity is clearly absent, and speed remains above 50 mph even for flows exceeding 2,000 vph. There is also little evidence that the second derivative of the speed-flow function is negative. Indeed, the upper branch of the speed-flow curve can be considered a bilinear function, with speed independent of flow in the low-flow range and a linear drop in speed in the moderate- and high-flow range. The speed drop observed is about 0.7 mph for every increase in flow of 100 vph, a little more than what Banks reported in San Diego (*9*).

The speed-flow plots also suggest that flows in excess of 2,200 vph are possible for 1-hr periods and certainly for 15-min and 2-min periods. This finding confirms previous observations in Canada and San Diego that flows substantially higher than the HCM capacity value of 2,000 pcphpl are possible even over a sustained period.

Effect of Different Lanes

The speed-flow relationships are plotted on the basis of the 15-min grouping for individual lanes and shown in Figure 4. Under low-flow conditions, there seems to be little difference in the speeds measured on the shoulder lane and those measured on the middle lane. As flow increases, the difference in speed becomes more apparent, with traffic on the shoulder lane moving at about 3 to 5 mph lower than that on the middle lane. Also, a higher value of maximum flow is recorded in the middle lane than in the shoulder lane. The difference in speeds and maximum flow attained in the two lanes may be

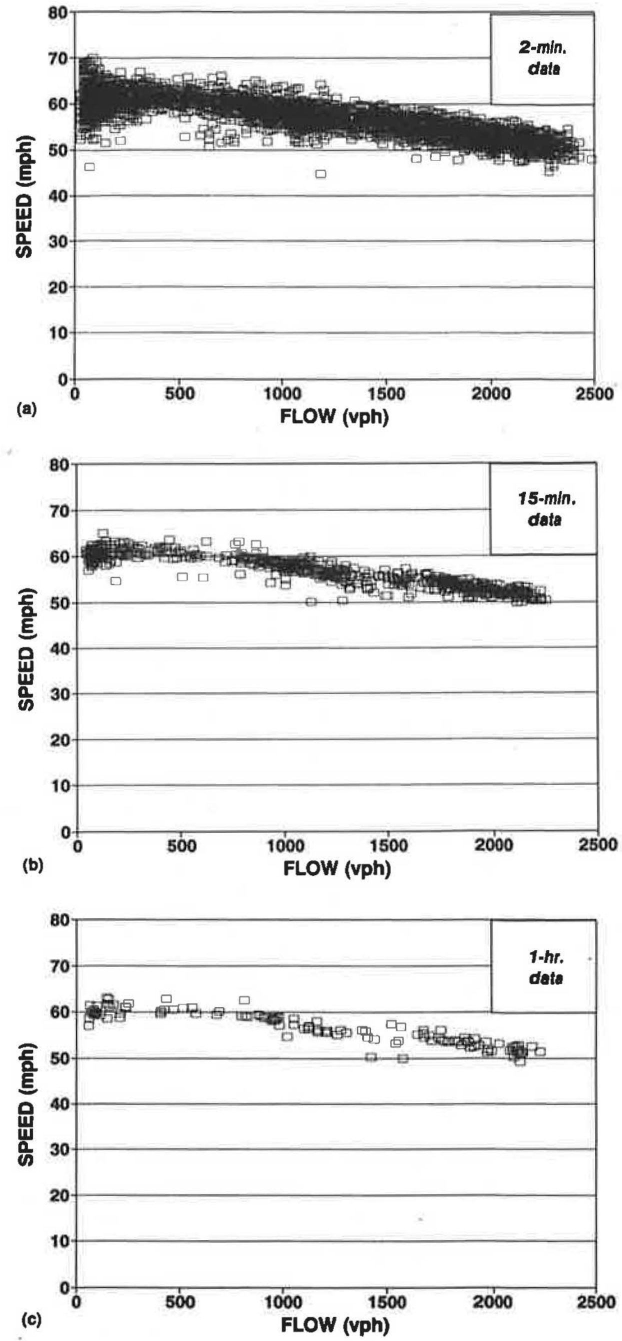


FIGURE 3 Speed-flow plot at Caldecott Tunnel for different sampling intervals.

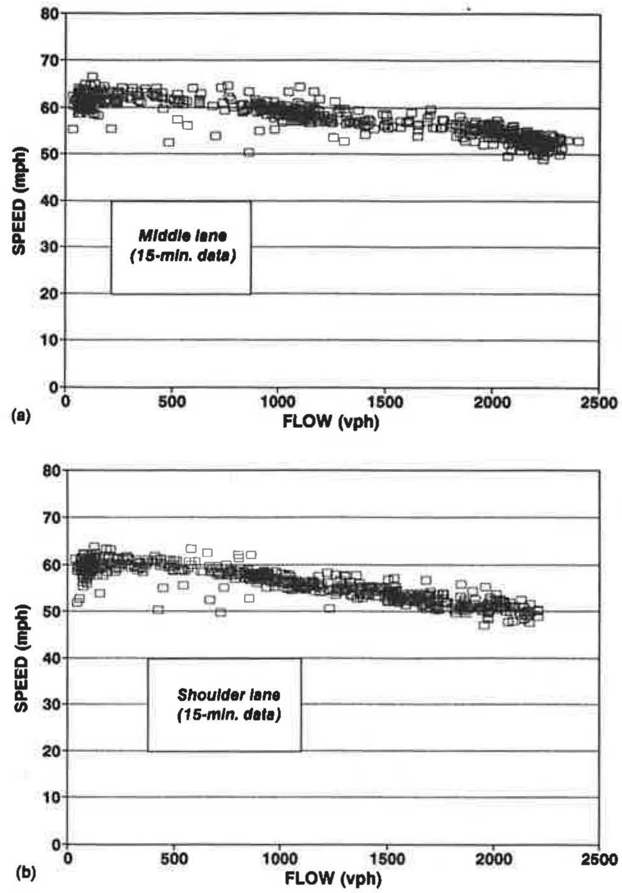


FIGURE 4 Speed-flow plot at Caldecott Tunnel for different lanes.

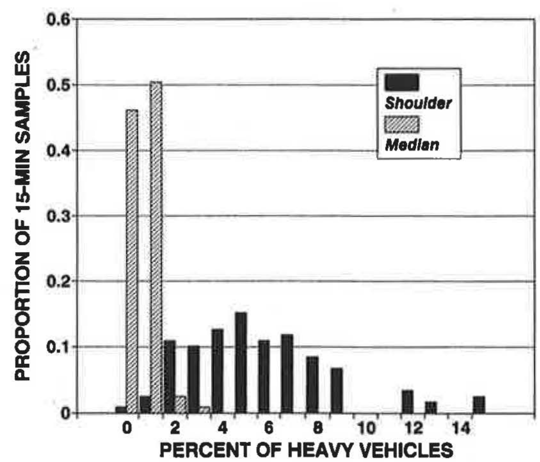


FIGURE 5 Distribution of heavy vehicles.

caused by a larger proportion of heavy vehicles and other slower moving vehicles in the shoulder lane.

Figure 5 shows the distribution of the heavy vehicles for the two lanes. Most of the time, the percentage of heavy vehicles is small, with 45 percent of the cases without heavy vehicles in the middle lane. On the other hand, there is a wide range in the proportion of heavy vehicles in the shoulder lane; in a number of cases, this proportion exceeds 10 percent. On average, 2.7 percent of heavy vehicles travel on this stretch of the roadway throughout the day, with the mean speed of heavy vehicles about 5 mph lower than that of the passenger cars.

Effect of Upstream Queue

The effect of an upstream queue on the speed-flow characteristics at the site is also investigated by isolating the speed-flow data corresponding to periods with an upstream queue. The precise times when the queue forms and vanishes are not known. However, in general, an upstream queue is present at the entrance of the tunnel during the morning peak hours.

Figure 6 shows speed-flow data in which cases with an upstream queue are plotted differently than those without an upstream queue. The data are gathered at 15-min intervals for the middle and shoulder lanes. Judging from the plots for both the lanes, there is no evidence that the speed-flow characteristics are any different when there is an upstream queue than when there is not. That the speed is not different between the two groups of data is not surprising because vehicles discharging from the queue would have returned to their normal speed at the end of the tunnel, which is about 3,000 ft long. The maximum flow attained when there is a queue is also comparable with the high flows attained otherwise. However, it is not clear why flows dropped significantly below the maximum possible on a number of occasions.

Comparison with HCM Curves for Freeways

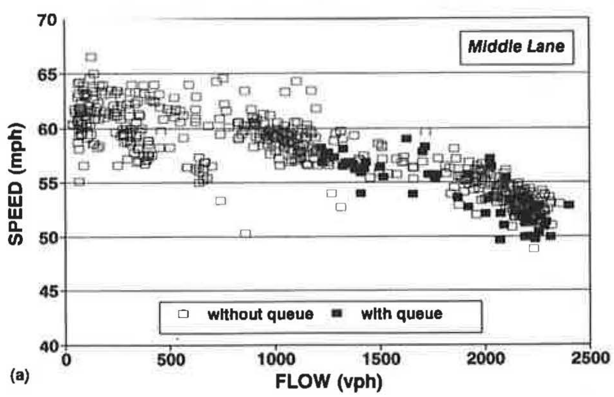
To compare the observed speed-flow data with the HCM curves, the speed, flow, and proportion of heavy vehicles must be grouped in 15-min periods in accordance to the time period of measuring flow used in the HCM. Because it is not possible to distinguish the different types of heavy vehicles from the data available, a single value of truck equivalent factor is used to convert the vehicle flows to passenger car flows. For downgrades, the equivalent factors given in the HCM are rather imprecise. The recommended procedure is to establish the average truck speed and to obtain the equivalent grade of the same length as outlined in Appendix I of Chapter 3 in the HCM (*2*). On the basis of an average truck speed of 50 mph, an equivalent factor of 2 is used. Because the lane width and lateral clearance of the site are ideal, no adjustment is made for these factors. In addition, no adjustment is made for the influence of driver population, which is assumed to be commuter traffic. Because the amount of adjustment varies for each 15-min period, adjustments are made on the data points rather than on the speed-flow curves. The adjusted 15-min speed-flow data are plotted in Figure 7, together with the HCM speed-flow curves corresponding to four- and eight-lane freeways with a design speed of 70 mph.

The observed data deviate from the HCM curves, especially under high-flow conditions. On a number of occasions, a flow in excess of the capacity value of 2,000 pcphpl has been observed. If an operational analysis of the freeway section were performed for various 15-min intervals on the basis of HCM curves, LOS E would be encountered on a large number of instances in which the speed is higher than the 55-mph speed limit.

The discrepancy between the observed data and the HCM curves may result simply because the site under investigation is not an average one represented by the HCM. On the other hand, the HCM recommended values used may not be appropriate. If the latter is true, then there are three possibilities in explaining the difference between the observed and predicted data. First, the pcu factor may be too high. Although possible, this error is difficult to judge. Moreover, even assuming a passenger car–unit value of 1.0, the difference between observed and predicted remains high. Second, the HCM capacity value of 2,000 pcphpl may be too low. This possibility was already noted in the previous section. Third, the speed-flow curve may be inappropriate. If the uncongested speed-flow data found in this site are representative, they indicate that the second derivative in the speed-flow curve is unlikely to be negative and is definitely not so negative that it results in a precipitous drop in speed near capacity.

Comparison with JHK Curves for Multilane Highways

The basis of comparing the speed-flow characteristics obtained from the Caldecott Tunnel site with the JHK method of analyzing multilane highways is that the freeway section can be regarded as a multilane divided highway with no access



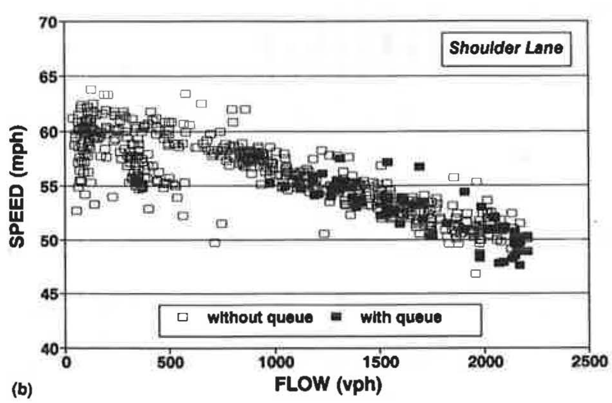


FIGURE 6 Speed-flow plot at Caldecott Tunnel with and without upstream queue.

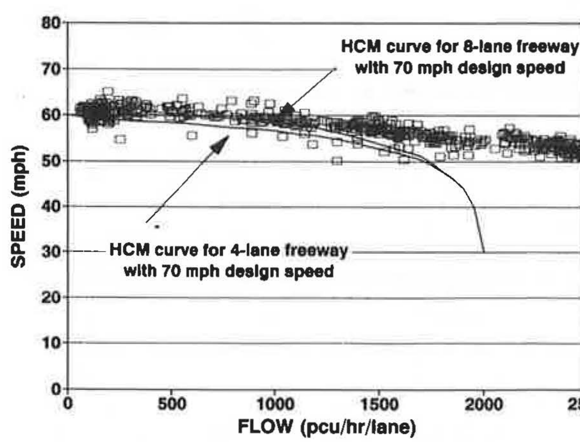


FIGURE 7 Comparison of speed-flow plot with HCM curves for freeways.

points. The number of differences between the current HCM method of evaluating freeways and the JHK method of evaluating multilane highways requires that the speed-flow data be adjusted differently. First, the speed of passenger cars is used in obtaining the speed-flow plot. For the set of data used, however, there is only a small difference between the passenger car speed values and the all-vehicle speed values. The method of equivalent grade is again used to estimate the heavy-vehicle factor, but in this case an equivalent factor of 1.7 is used as recommended.

In selecting the appropriate speed-flow curve, the JHK method requires that the free-flow speed of passenger cars in the section be known. Two ways of determining the free flow speed have been suggested. The first is to estimate the ideal free flow speed from the value of the posted speed limit and then to reduce the speed value by adjustments for nonideal geometric conditions. The second requires a site measurement of passenger-car speeds under low- or moderate-flow conditions.

The posted speed limit on this freeway is 55 mph. According to Table 7-2 in the JHK procedure (*5*), this limit gives a corresponding ideal free flow speed of 59.3 mph. Because the geometric conditions are ideal at this site, no reduction in the free-flow speed value is necessary. The speed-flow curve that is based on this value, together with the adjusted speed-flow data, is shown in Figure 8. The JHK speed-flow curve marginally underestimates the speeds under low-flow conditions but overestimates the speeds under moderate- and high-flow conditions. In general, this speed-flow curve gives a better reflection of the conditions at the Caldecott Tunnel site than the HCM speed-flow curves used in the previous section. However, a maximum flow of 2,300 pcphpl attained at a speed of about 50 mph is still higher than the JHK capacity value of 2,200 pcphpl at 55 mph. Furthermore, the speed at capacity is approximately 10 mph lower than the free flow speed, compared with a speed reduction of 5 mph as suggested in the JHK method.

If speed measurements are taken from the field to estimate the free flow speed and a 15-min sample is used, then the speed value in any of the data points falling within the low- or moderate-flow region shown in Figure 8 could have been used. In this case, the free flow speed may range from 55 to 63 mph, and any speed-flow curve with intercepts within that range may have been chosen for the analysis. Figure 9 shows the envelope of the speed-flow data, together with the curves corresponding to the upper and lower values of the measured free flow speed. There can be quite a difference in the choice of the curve used. However, the difference may not be significant in the evaluation of the level of service.

CONCLUSION

It could be argued that the results discussed in the previous paragraphs are limited because only a single location was investigated. Certainly, some of the findings cannot be generalized and can only be confirmed with observations from other sites. Nevertheless, the findings not only gave some insight into the relationship between speed and flow but should also provide motivation for more investigations on the speed-flow interaction of traffic on the highways.

The manner in which the speed and flow data were gathered not only resulted in a sufficiently large number of samples for analysis but also ensured reliable measurement of traffic speeds. The results indicate that, under uncongested conditions, there is a well-defined relationship between speed and flow that can be described in the form of a bilinear function. The study confirms earlier reports that there is no precipitous drop in speed as flow approaches capacity and that flows in excess of 2,200 pcphpl are possible. However, the presence of an upstream queue does not seem to affect the speed-flow relationship at the location studied.

The results also show that the speed-flow conditions at the Caldecott Tunnel have not been well represented by the HCM speed-flow curves but were reasonably reflected by the JHK speed-flow curves for multilane highways. If the results obtained are representative, it may be better to evaluate freeway basic sections by treating them as multilane divided highways with no access points and following the JHK procedure for multilane highways.

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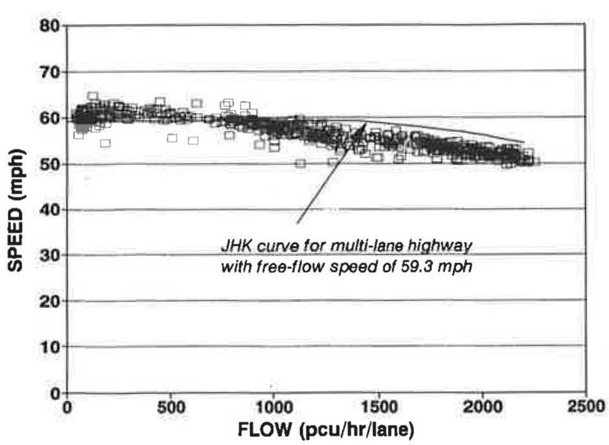

FIGURE 8 Comparison of speed-flow plot and JHK curve on the basis of posted speed limit.

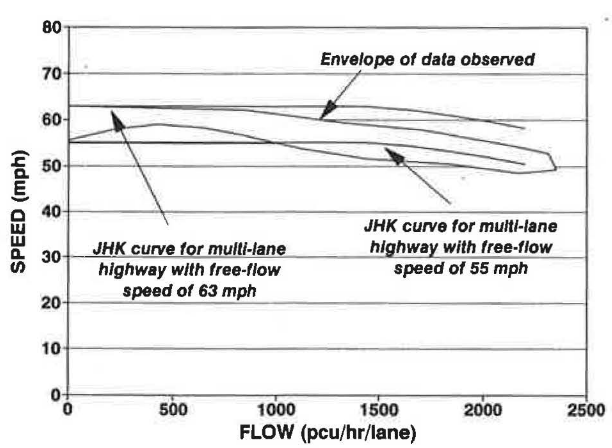

FIGURE 9 Comparison of speed-flow plot and JHK curve on the basis of estimated free flow speed.

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REFERENCES

1. *Special Report 87: Highway Capacity Manual 1965*. HRB, National Research Council, Washington, D.C., 1965.
2. *Special Report 209: Highway Capacity Manual 1985*. TRB, National Research Council, Washington, D.C., 1985.
3. J. S. Drake, J. J. Schoefer, and A. D. May. Statistical Analysis of Speed-Density Hypothesis. *Proc., 3rd Symposium on the Theory of Traffic Flow*, Elsevier, N.Y., 1967.
4. V. F. Hurdle and P. L. Datta. Speeds and Flows on an Urban Freeway: Some Measurements and a Hypothesis. In *Transportation Research Record 905*, TRB, National Research Council, Washington, D.C., 1983, pp. 127–137.
5. W. R. Reilly, D. W. Harwood, J. M. Schoen, and M. F. Holling. *Capacity and Level of Service Procedures for Multi-Lane Rural and Suburban Highways*. JHK & Associates and Midwest Research Institute, May 1989.
6. B. L. Allen, F. L. Hall, and M. A. Gunter. Another Look at Identifying Speed-Flow Relationships on Freeways. In *Transportation Research Record 1005*, TRB, National Research Council, Washington, D.C., 1985, pp. 54–64.
7. F. L. Hall and M. A. Gunter. Further Analysis of the Flow-Concentration Relationship. In *Transportation Research Record 1091*, TRB, National Research Council, Washington, D.C., 1986, pp. 1–9.
8. B. N. Persaud and V. F. Hurdle. Some New Ideas that Challenge Some Old Ideas About Speed-Flow Relationships. In *Transportation Research Record 1194*, TRB, National Research Council, 1988, pp. 191–198.
9. J. H. Banks. Freeway Speed-Flow-Concentration Relationships: More Evidence and Interpretations. In *Transportation Research Record 1225*, TRB, National Research Council, Washington, D.C., 1989.
10. F. L. Hall and L. M. Hall. Capacity and Speed-Flow Analysis of the QEW in Ontario. In *Transportation Research Record 1287*, TRB, National Research Council, Washington, D.C., 1990.
11. F. L. Hall and B. N. Persaud. An Evaluation of Speed Estimates Made with Single Detector Data from Freeway Traffic Management Systems. In *Transportation Research Record 1232*, TRB, National Research Council, Washington, D.C., 1989.

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