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53 Traffic Control and Operations

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Foreword

The problems associated with the construction of additional miles of freeways in urban areas continue to grow. The engineering problems ordinarily experienced are taking lower priority, while community concerns such as the displacement of persons and aesthetics gain more attention. Many a city finds that it has built its last significant amount of freeway mileage. Growing urban areas are, in many cases, securing additional transportation by substituting modes other than the automobile, and by making better use of the facilities that are already available. While waiting for rail or subway systems to be completed, many cities are trying to operate existing street and freeway systems in a more efficient manner.

For many years, the Committees on Freeway Operations, Highway Capacity, and Quality of Traffic Service of the Highway Research Board have been concerned with better traffic operations on freeways. This RECORD presents four papers, several discussions, and two abridgments on the subject, all of which were developed by these committees and presented at the Board's 47th Annual Meeting. Those concerned with the growing problems of making better use of freeways and wanting to know of the latest research on the subject will find the material to be pertinent. Highway department administrators and design, traffic, and maintenance engineers will find much of interest in these reports.

California has studied the possibility of enhancing freeway operations by establishing varying minimum speed limits on individual lanes. It was found that instead of the higher average speeds hoped for, lower speeds were actually experienced. Based on these findings, the first paper concludes that minimum speeds by lanes cannot be recommended.

The second paper, by two California researchers, presents findings derived from a study of the operations of a heavily traveled Los Angeles interchange area. Painted channelization was used to make operational changes, and aerial photography analysis was used to measure the effects. The researchers found that striping did enhance the operations, but if it is done wrong, it can have a detrimental effect. It was also found that aerial photography is a good way to measure the effects of such operational changes.

Three Texas researchers have studied the problems of capacity at merging areas, and their findings are presented in the next paper. Using the gap acceptance mode of control and gap distribution data, the researchers describe a new approach to the problem and demonstrate practical application of their work to design. Freeway designers using this concept will be able to evaluate various alternatives more rationally and choose the best.

The fourth paper presents economic criteria that prove the value of building a diamond interchange on a rural expressway at grade where traffic volumes have outgrown stop sign control. The alternatives generally employed in such situations (signals or fourway stop control) are shown to be more costly when economic parameters are used as measurements. The tables developed by the authors can easily be used by other agencies that have the same dilemma and wish to evaluate the alternatives.

The RECORD closes with abstracts of two papers that evaluate aspects of the freeway surveillance system operated on Detroit's Lodge Freeway. Texas Transportation Institute researchers report on progress on this project. Because the work was performed under NCHRP sponsorship and will be available from that publication series, only abstracts are presented.

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Minimum Speed Limits on Freeways

NORMAN C. WINGERD, Assistant Traffic Engineer, California Division of Highways

This study was made to determine the feasibility of establishing minimum speed limits by lane on multiple-lane highways. This was done by erecting signs and observing traffic at four different sites throughout the state. The factors under particular consideration included mean speeds, speed distributions, headways, headway distribution, volume distribution by lane, lane changing, passing on the right and enforcement problems. Before-and-after observations revealed few if any beneficial results of the minimum speed limits and showed some results definitely unfavorable to operation and safety.

•THE 1965 Regular Session of the California State Legislature requested the Department of Public Works to make a study of the feasibility of establishing minimum speed limits on multiple-lane highways on a lane-to-lane basis.

Based on historical evolution of driving habits and road development, it is customary to think that on a 4-lane road the right lane in either direction is the driving lane and the left lane is the "passing" lane. However, when traffic flow reaches a certain level, many drivers stay in the left lane instead of returning to the right lane between each vehicle overtaken, because otherwise they would be weaving back and forth continually. This annoys drivers who desire to go even faster than the passing vehicles that stay in the left lane.

Another less frequently observed problem is that some drivers drive in the left lane at speeds less than the speed limit even when there is very little traffic in the right lane. This again requires fast drivers to change lanes and pass on the right, which, although legal, is thought undesirable.

If signing or other traffic control measures are to have a beneficial effect, we would expect some of the following changes to occur:

1. The speed distribution for a given lane should fall into a more uniform grouping. This would be indicated by a smaller variation of speeds. Yet, this should be accomplished without reducing the average speed because a reduction in average speed would automatically result in greater impedance to the faster group of drivers.
2. The distribution of headways is also a significant indicator. We would expect to find fewer platoons of vehicles, and smaller numbers of vehicles in them. A beneficial effect would evidence itself by a decrease in the number or percentage of short headways (less tailgating).
3. The number of lane-changing maneuvers would be expected to decrease.
4. We would hope to find fewer vehicles passing on the right.
5. If slower vehicles were moved to the right, we would expect volumes to redistribute throughout the lanes, and generally increase in the right lanes while decreasing in the left lanes.

STUDY PROCEDURE

Four study sites (Fig. 1) were selected throughout the state freeway system: one 4-lane, one 6-lane, one 8-lane, and one where an 8-lane freeway narrows to 6 lanes. The

Paper sponsored by Committee on Freeway Operations and presented at the 47th Annual Meeting.

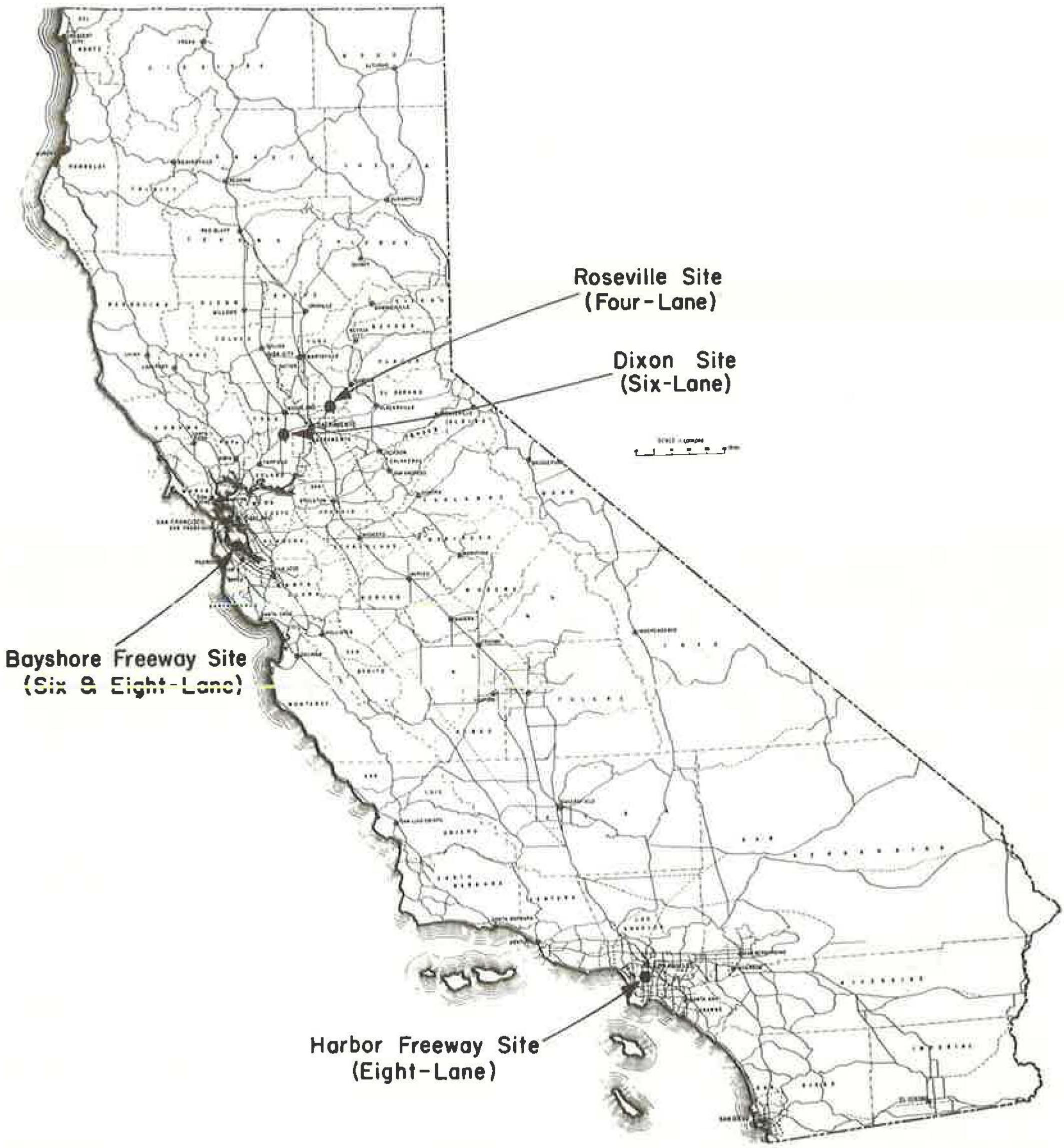

Figure 1. Location of minimum lane speed study sites.

latter actually served as two study sites. The 4-lane section was on Interstate 80 (Roseville Freeway) near Roseville between South Roseville Road and Foothill Farms pedestrian overcrossing. The 6-lane section was on Interstate 80 near Dixon between Pedrick Road and Dixon-Grant Road. The 8-lane section was on Route 11 (Harbor Freeway), between 111th Place and 149th Street in Los Angeles. The combined section was on US 101 (Bayshore Freeway) in San Mateo County between Peninsular Avenue and Ralston Avenue. These sites were selected primarily because (a) they have nearly straight alignment, (b) they have no sustained grades that would significantly affect truck speeds, (c) they have no high-volume on- or off-ramps that would induce an excessive amount of lane changing, passing on the right, and below-normal speeds, and (d) they all have overcrossings on which signs could be mounted. It is noted that the first three considerations create an atmosphere for high speeds. It was not thought desirable to impose restrictive minimum speeds where there are slower design speed features.

At each of the study sites, signs were erected over the lanes of traffic imposing minimum speeds by lane (Fig. 2). Each study site had a minimum of three sets of signs at approximately 1-mile spacing over a 3- to 4-mile length of roadway. The signs were white letters on a black background and each sign was 7 by 9 ft in dimension, which is almost as large as possible for a sign that is identified with one 12-ft lane.

"Before" and "after" data were gathered at each of the study sites. The "after" data were taken after the signs had been in place a minimum of 2 weeks. Speed, volume, and headway information was obtained with the Bureau of Public Roads Traffic Analyzer (1), a recording device that prints numbers on a paper tape showing the speed of each vehicle across a 36-ft speed trap, as well as its time of day to the nearest ⅓ second. The Traffic Analyzer data were taken near the downstream signs at each study site. Observations of more than 85,000 vehicles were made with this equipment for the purpose of this study. The timing device of the Analyzer was frequently checked and calibrated by numerous comparisons with the calibrated speedometers of Highway Patrol cars through the trap. Stopwatch observations over a longer trap length were also used for comparison.

The lane-changing incidence and the incidence of passing on the right was obtained by a visual count and also from an analysis of time-lapse photography using 16-mm movie film taken at 1 frame per second.

To gain some knowledge of the enforcement problems involved, a sampling of information was taken from the violators of the minimum speed, i.e., drivers who were stopped for driving slower than the posted speed for the lane involved. This information was gathered with the cooperation of the California Highway Patrol and the Los Angeles Police Department.

Figure 2. Placement of minimum speed signs on 8-lane freeway.

SPEEDS

The maximum speed limit in California is 65 mph for 2-axle vehicles and buses (except for a few sections of freeway where the maximum speed is 70 mph) and 50 mph for vehicles and combinations with 3 or more axles. Sections of highway which have a 65-mph speed limit may be zoned for a maximum limit less than 65 on the basis of an engineering and traffic survey. These lower limits are "prima facie" and may be exceeded if the driver can establish by competent evidence that speed in excess of the prima facie limit does not violate the basic speed law. To comply with the basic speed law, a driver shall not drive at a speed greater than is reasonable or prudent having due regard for weather, visibility, traffic, and surface and width of the highway, and in no event at a speed which endangers the safety of persons or property.

In addition to these maximum speed limits, California has a minimum speed law which states that no person shall drive at such a slow speed as to impede or block the normal and reasonable movement of traffic except when reduced speed is necessary for safe operation, or because upon a grade, or in compliance with law. Whenever the Department of Public Works determines on the basis of an engineering and traffic survey that slow speeds consistently impede the normal and reasonable movement of traffic, the Department may determine and declare a minimum speed limit. The vehicle code further provides that official signs may be erected directing slow-moving traffic to use a designated lane. It is the policy of the Division of Highways to install "Slower Traffic Keep Right" signs at approximately 5-mile intervals on divided highways.

In 1965 the California Highway Patrol maintained an aggressive enforcement program toward those vehicles that failed to drive in the right-hand lanes and/or impeded the normal flow of traffic. During this period the Patrol issued 21,783 arrests for these violations.

At very high traffic volumes, such as those found during peak hours on many urban freeways, the maximum attainable speed is controlled by the presence of the other vehicles that "got there sooner," and any minimum speed limit would have no meaning because it would be so much greater than the maximum attainable speed. At lower volumes, cars that catch up to a slow car in the left lane (or lanes) will pass on the left or right if possible. If they are unable to pass because cars in the adjacent lanes are traveling approximately the same speed, they will queue up behind the slow car and maintain approximately the same average headway as before queueing up. Even though the rate of flow of a lane is not severely reduced by a slow car, the speeds of all the cars in the queue are reduced and thus all travel times are increased. Drivers are very conscious of increased travel times and this can be quite a source of irritation to them.

The problem of slow drivers in the left lane(s) is most severe when there are only 2 lanes in one direction, since a slow driver can "block" the highway if he drives about the same speed as a vehicle in the right lane. On a 6- or 8-lane facility the chances of 3 or 4 drivers driving side by side at the same speed are very small.

It is conceivable that slow-moving vehicles can create an accident problem both by driving slow in the left-hand or fast lanes and by causing faster vehicles to use the right-hand or slow lane. The only problem of this nature that has come to our attention is caused by slow trucks on long grades where fast vehicles in the right lane occasionally run into the rear of a slow truck. In this case, the trucks are traveling very slow, sometimes only 10 mph. It would require additional very specific legislation to make it illegal to go slower than a speed that cannot be attained by large numbers of vehicles on grades.

It was initially thought that the posted minimum speeds should be at least 15 mph below the posted maximum speed limits. This would mean that left lanes of traffic would be posted at a minimum speed of 50 mph or less, and the right lanes, used by trucks, would be posted at 35 mph or less. However, after a consideration of existing spot speeds, it was realized that the posted minimum speeds would have to be much higher than this to have any effect. For the purpose of this study, the posted minimum speeds used for a 2-lane section were 60 and 45 mph for the median and shoulder lanes respectively. For the 3-lane sections, 60, 55, and 45 mph minimums were used from left to right lane respectively. At the 4-lane sites, 60, 60, 55, and 45 mph minimums were used, proceeding from left to right lane. Lane designations throughout this report are by number, increasing from left to right.

In the left lane (lane No. 1) of traffic a driver was faced with a 5-mph driving range, i.e., a minimum speed of 60 mph and a maximum speed of 65 mph. A truck driver in the right lane of traffic was faced with a minimum speed of 45 mph and a maximum speed of 50 mph. By posting the second lane from the right at a speed greater than 50 mph, trucks were legally prohibited from using any lane other than the extreme right one. If minimum speeds by lane were posted on a widespread basis, the minimum speed in the lane adjacent to the right lane would have to be lowered to accommodate trucks passing slower vehicles, and therefore the signs would be of little significance to other vehicles. Or provisions could be provided in the vehicle code to exempt trucks from the minimum speed restriction when in the process of passing slower vehicles.

To our knowledge no minimum speed limits have been established previously for each lane of multiple-lane highways. Several states have statewide minimum speed limits, or minimum speed limits on certain stretches of highway, but these apply to the entire roadway uniformly, rather than individually by lane. Very little research has been published, and it is of a qualitative nature rather than quantitative.

Mean Speed and Deviation

Since a reduction of delay is a major objective of minimum speeds, the comparison of "before" and "after" mean speeds and deviations is important. The "before" and "after" observations were of the same period of the day, the same character of traffic and almost identical rates of flow. The minimum number of speed observations at any one site was greater than 2000 vehicles and included all vehicles. Neither the "before" nor the "after" data were used for any periods when abnormal conditions (parked vehicles on shoulder, etc.) existed. To insure valid speed comparisons, all data were collected when volumes were well below capacity, the maximum being approximately 1300 vehicles per hour for any one lane. These comparisons at all of the study sites are given in Tables 1, 2, 3, 4 and 5. Speed distribution curves are shown in the Appendix (Figs. 11 through 25). The mean speeds are calculated as the numerical average of the spot speed of each vehicle.

Vehicles travel at variable rates of speed past a given point. There is a spread, or dispersion, of speeds about the mean. A statistical measure of this dispersion is called the standard deviation: 68 percent of the vehicles will travel within one standard deviation (plus and minus) of the mean, and 95 percent of the vehicles will travel within two standard deviations.

Positive reactions to the minimum speed signing would reveal themselves in one or both of two ways relative to speed: (a) the mean speed would increase; (b) the standard deviation would decrease. The reverse of these results would be considered to accomplish the reverse of what minimum speed limits are intended to do; i.e., there would be more interference by slow vehicles with the desired speed of the faster vehicles.

Table 1 would indicate that at the suburban site where there were only 2 lanes in each direction, the mean speed did increase in both lanes. The standard deviation, however, showed an increase in the left lane while it showed a decrease in the right lane. This was the only study site that showed a significant positive change in speeds. This positive change, however, does not necessarily reflect an improvement in traffic operations. A look at the speed distribution curves for lane 1 (shown in Fig. 11) reveals that there was little change in speeds of vehicles traveling less than 68 mph. The vehicles in the median lane traveling faster than 68 mph

TABLE 1

EFFECT OF MINIMUM SPEED SIGNING ON SPEEDS—I-80 AT FOOTHILL FARMS PEDESTRIAN OVERCROSSING

Lane No.	Mean Speed (mph)		Standard Deviation (mph)		Significance of Diff. in Speed at 95% Level*
	Before	After	Before	After	
1	67.2	68.2	5.30	5.64	S
2	59.0	59.6	7.11	6.92	S

* S indicates significant; NS indicates not significant in all Tables.

TABLE 2

EFFECT OF MINIMUM SPEED SIGNING ON SPEEDS—I-80 AT DIXON

Lane No.	Mean Speed (mph)		Standard Deviation (mph)		Significance of Diff. in Speed at 95% Level
	Before	After	Before	After	
1 (Weekday)	70.5	68.6	5.27	5.34	S
2 (Weekday)	65.7	63.7	5.75	5.84	S
3 (Weekday)	57.8	54.2	7.92	7.30	S
1 (Sunday)	69.6	67.6	5.11	4.37	S
2 (Sunday)	64.7	63.0	5.20	5.16	S
3 (Sunday)	57.8	57.6	7.01	6.69	NS

TABLE 3

EFFECT OF MINIMUM SPEED SIGNING ON SPEEDS—BAYSHORE FREEWAY AT SUNNYBRAE AVENUE

Lane No.	Mean Speed (mph)		Standard Deviation (mph)		Significance of Diff. in Speed at 95% Level
	Before	After	Before	After	
1	67.7	67.9	5.21	4.72	NS
2	63.1	64.0	5.07	4.63	S
3	59.7	*	6.00	*	*
4	51.6	51.5	7.15	6.67	NS

* Insufficient data.

TABLE 4

EFFECT OF MINIMUM SPEED SIGNING ON SPEEDS—BAYSHORE FREEWAY AT RALSTON AVENUE

Lane No.	Mean Speed (mph)		Standard Deviation (mph)		Significance of Diff. in Speed at 95% Level
	Before	After	Before	After	
1	67.5	67.5	4.36	4.74	NS
2	61.7	61.2	5.29	5.35	NS
3	52.6	52.8	6.98	6.97	NS

TABLE 5

EFFECT OF MINIMUM SPEED SIGNING ON SPEEDS—HARBOR FREEWAY AT 149th STREET

Lane No.	Mean Speed (mph)		Standard Deviation (mph)		Significance of Diff. in Speed at 95% Level
	Before	After	Before	After	
1	67.6	67.2	4.45	4.13	S
2	67.8	67.5	5.12	4.43	S
3	60.6	60.8	5.86	5.70	NS
4	54.5	54.3	7.19	6.85	NS

speeded up, and thereby increased the mean speed as well as deviation. Minimum speed signs were not intended to increase speeds of the vehicles that are already exceeding the speed limit.

At the I-80 site near Dixon, which was 3 lanes in each direction at comparatively low traffic volume and density, a significant decrease occurred in mean speeds in almost all lanes. This happened during weekday study and Sunday study as well. Since there was no concomitant reduction in variability of speeds among cars in a lane, it may be said that the minimum speed signing accomplished the opposite of what was intended. Overall travel time was increased, and presumably the fast drivers who most desire minimum speeds were forced to go slower. It will be noted later that at this high-speed 3-lane rural study site there was a definite redistribution of traffic among lanes. This redistribution is considered to contribute to the slowdown in mean speeds; i.e., some of the slow drivers moved into the fast lanes and thereby caused a reduction in speed of all drivers. In other words, they interfered.

Another 3-lane site used for study was on the Bayshore Freeway near Ralston Avenue. Table 4 shows that there was no significant difference in speeds after minimum lane speed signs were imposed. The slight increase in standard deviation would be considered of negative value, if it were large enough to be significant.

At the 4-lane study site on the Bayshore Freeway, Table 3 shows little significance to any changes in mean speeds. The standard deviation was reduced slightly. However, there was a second external factor that probably affected the "after" data at this site. Some necessary construction had the shoulder lane closed for several days during the week prior to the collection of "after" data, and some downstream ramps were permanently closed. A reduction of volume in the right lane is evidence of this fact.

The Harbor Freeway site, with speed results given in Table 5, is considered to be more representative of a four-lane metropolitan site. The Harbor Freeway site showed some mean speed change in lanes 1 and 2 (the left lanes of 4 in one direction). Due to the large samples of the traffic observed at this site, even the small change is considered statistically significant at the 95 percent level. This change was a reduction in mean speed of about 0.4 mph, not enough to say that safety was increased, and in the opposite direction to the intent of minimum limits. Mean speeds in lanes 2 and 3 showed a small change, but they are not considered significant. All lanes showed some improvement by reducing the standard deviation. The overall effect of minimum speed signing at this site would not reduce travel times.

TABLE 6

MINIMUM SPEED VIOLATION RATE—I-80 AT FOOTHILL FARMS PEDESTRIAN OVERCROSSING

Condition	Lane 1: Percent of Vehicles Traveling Less Than 60 mph	Lane 2: Percent of Vehicles Traveling Less Than 45 mph	All Lanes
Before	7.0	2.8	4.19
After	6.0	1.6	3.56

Observation of more than 8,000 vehicles.

Minimum Speed Violation

Tables 6, 7, 8, and 9 show the effect of minimum speed signing on the "violation rate." This rate is defined as the number of minimum speed violations per 100 vehicles passing the study site. Even though there was no actual violation in the "before" condition, it is referred to as such in this report.

Table 6 (the 1-directional 2-lane study site near Roseville) shows a slight decrease in the violation rate for both lanes of travel.

All of the 3-lane study sites, as shown in Tables 7 and 8, indicate an increase in the violation rate.

The violation rate of the Bayshore Freeway at Sunnybrae Avenue is not shown because construction operations near the site made observations abnormal.

Table 9, which gives the minimum speed violation rate at the Harbor Freeway site, shows that there was a slight increase in the violation rate for lanes 1 and 2 together with a decrease in violation for the 2 right lanes.

When comparing the minimum speed violation rates (Tables 6, 7, 8, and 9) with the volume distribution (Figs. 7, 8, 9, and 10) it may be noted that where the total volume was light, the minimum speed signs resulted in a shift to the left; i.e., the percentage of vehicles traveling in the left lanes increased. This in turn increased the incidence of violation in all lanes, because a slow vehicle staying in the right lane may not have been a violator at all. All of the 3-lane sites showed this to be the case.

HEADWAYS

A headway is defined as the time interval between passage of consecutive vehicles moving in the same direction past a given point. In this report, as in most uses, it

TABLE 7

MINIMUM SPEED VIOLATION RATE—I-80 AT DIXON

Condition	Lane 1: Percent of Vehicles Traveling Less Than 60 mph	Lane 2: Percent of Vehicles Traveling Less Than 55 mph	Lane 3: Percent of Vehicles Traveling Less Than 45 mph	All Lanes
Weekday, Before	0.5	3.2	4.0	2.84
Weekday, After	3.8	6.3	11.0	7.03
Sunday, Before	2.0	2.5	3.2	2.49
Sunday, After	3.0	5.0	3.0	3.67

Observation of more than 16,000 vehicles.

TABLE 8

MINIMUM SPEED VIOLATION RATE—BAYSHORE FREEWAY AT RALSTON AVENUE

Condition	Lane 1: Percent of Vehicles Traveling Less Than 60 mph	Lane 2: Percent of Vehicles Traveling Less Than 55 mph	Lane 3: Percent of Vehicles Traveling Less Than 45 mph	All Lanes
Before	3.6	7.9	10.0	6.68
After	4.7	14.3	10.8	8.53

Observation of more than 16,000 vehicles.

TABLE 9

MINIMUM SPEED VIOLATION RATE—HARBOR FREEWAY AT 149th STREET

Condition	Lane 1: Percent of Vehicles Traveling Less Than 60 mph	Lane 2: Percent of Vehicles Traveling Less Than 60 mph	Lane 3: Percent of Vehicles Traveling Less Than 55 mph	Lane 4: Percent of Vehicles Traveling Less Than 45 mph	All Lanes
Before	2.3	3.0	16.0	8.0	7.69
After	2.7	4.4	13.0	6.3	5.13

Observation of more than 25,000 vehicles.

refers to a single lane of travel. An analysis of headways is important because short headways (tailgating) are of great concern from the standpoint of highway safety.

A certain proportion of all vehicles on the road travel in platoons, often with short headways, even when traffic volume is very low. There are several reasons why this occurs. One reason is that when there is a reasonable variability in speeds (by definition, this is one condition of free flow) the headway in front of a given vehicle is continually changing, and often approaches a minimum just before a passing maneuver takes place. Thus, sheer chance (probability) will account for many short headways as well as some very long ones. Another reason is that when one vehicle is traveling considerably below the average speed (for example, a car with a house trailer required by law to travel slowly), all the other vehicles on the road must pass it. These other vehicles are normally scattered among all the available lanes, but when they pass the slow vehicle they are compressed into a roadway of one less lane. In the case of a 4-lane freeway, this means that all the vehicles except the slow one have to use 1 lane instead of 2 while passing the slow vehicle. This causes the headways in the "passing lane" to become very short. A third reason for short headways, and probably not very important, is that some drivers just like to drive with short headways.

Finally, platoons can be formed because the lead car is going slow and other cars catch up with it when there is no opportunity to pass on the right. (This is closely related to the second reason given above.) It is this type of platoon that minimum speed limits are designed to alleviate.

Headways between vehicles are tied to traffic volume by mathematical laws. Specifically, headway (unit time/vehicle) is the inverse of volume (vehicles/unit time). If the road has plenty of capacity, the traffic volume represents the demand. When the demand is 720 vehicles per hour per lane, the average headway is 5 seconds; i.e., there are 3600 seconds in one hour and if there are 720 headways, the average is 3600 divided by 720, or 5. If these 720 cars all came along in one continuous platoon, at an average headway of 2 seconds, there would be 719 headways of 2 seconds apiece and one headway of 2162 seconds to take up the rest of the hour. This would be very undesirable operation. It would be more desirable to reduce the number of very short headways and increase the number of longer headways (the total number has to remain 720), but of course this would shorten each "long" headway. Generally, the more platooning there is, the more very short headways will occur, and the less desirable traffic operations will be.

Figures 3, 4, 5, and 6 show the percentage of very short headways before and after erecting minimum speed limit signs at the four locations studied. At the Roseville site (Fig. 3), it will be seen that at 1000 vph, headways of less than 1.5 seconds were increased from 9 percent before to 11 percent after the signs were erected. This is an increase of about 20 percent. The headways less than 2 seconds also increased at this site.

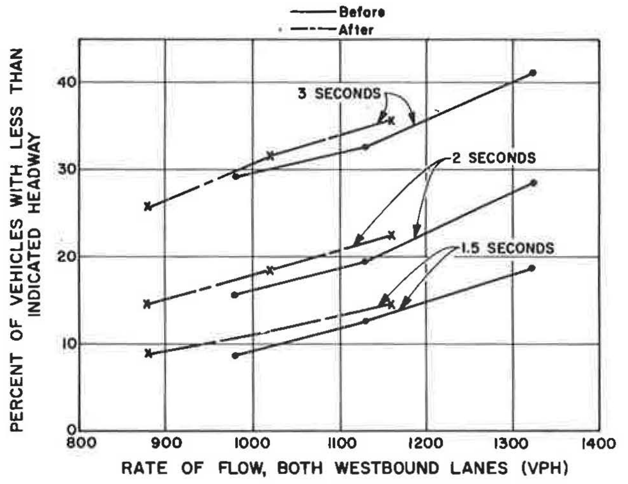

Figure 3. Effect of minimum lane speed signs on short headways—I-80 near Roseville.

At the Dixon site (Fig. 4), the percentage of very short headways also increased at most rates of flow. At the Bayshore and Harbor sites, total flow was much heavier, and the percentage of very short headways was not affected significantly because they were more a result of sheer mathematical chance than anything else.

Although the increase in very short headways caused by the minimum speed signs was not great, there was an increase, especially at low volumes, and it must be concluded that the signs did not accomplish the purpose of reducing platooning behind slower vehicles. This finding is consistent with the finding previously described, of a shift to the left by slower vehicles.

VOLUME DISTRIBUTION BY LANE

Figures 7, 8, 9, and 10 show the effect of the signing on the traffic distribution by lane. The percent of total volume both "before" and "after" is shown.

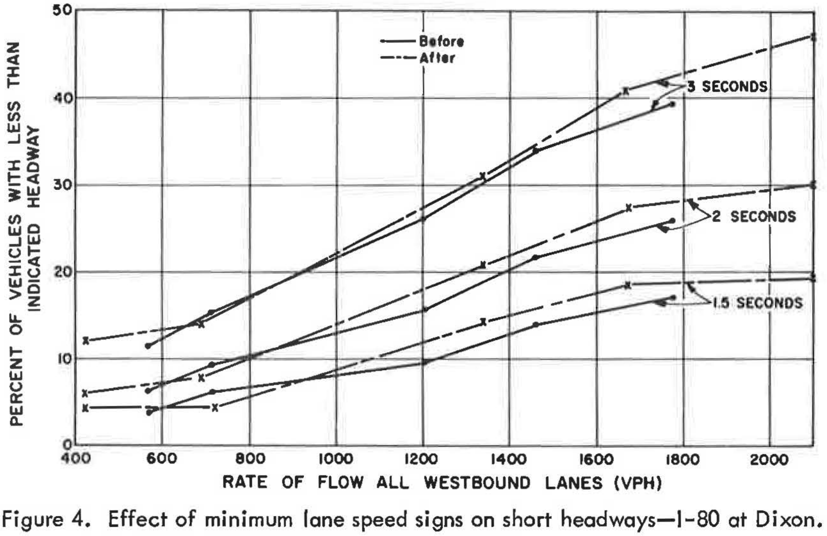

Figure 4. Effect of minimum lane speed signs on short headways—I-80 at Dixon.

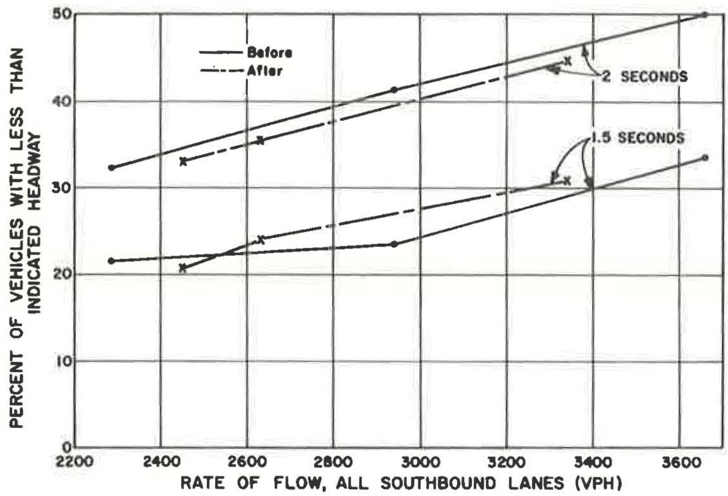

Figure 5. Effect of minimum lane speed signs on short headways—Bayshore Freeway at Ralston Avenue.

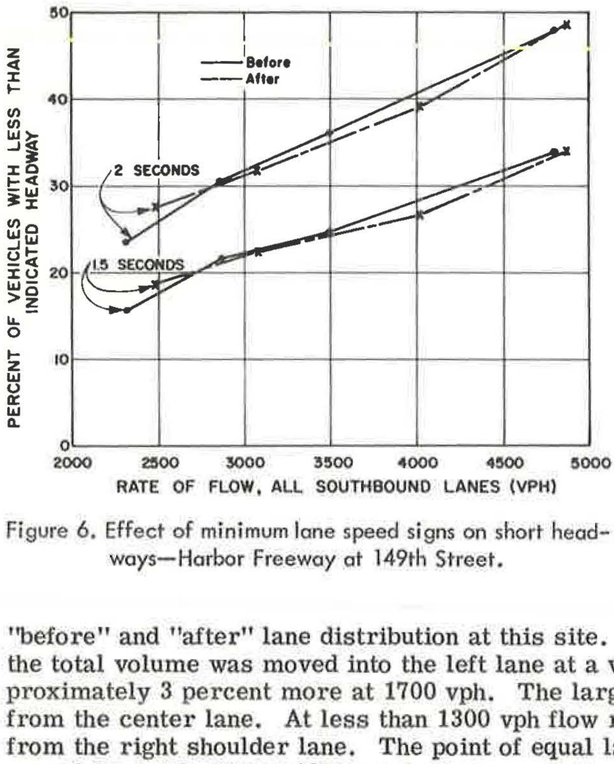

Figure 6. Effect of minimum lane speed signs on short headways—Harbor Freeway at 149th Street.

At the 1-directional 2-lane study site, the range in observed volumes varied through the study period from approximately 800 vph to 1300 vph (total for both lanes). Figure 7 shows that the minimum speed signing did redistribute the vehicles throughout this flow rate. This redistribution did not move slow vehicles to the right, as was originally anticipated, but instead, moved approximately 3 percent more of the total vehicles into the left lane.

The 1-directional 3-lane study site at Dixon also showed a definite redistribution of vehicles in the "after" period. The volume range during the study period was approximately 400 to 700 vph (all lanes) on weekdays, and from 1200 to 2100 vph (all lanes) on Sunday. Figure 8 shows the relationship of "before" and "after" lane distribution at this site. Approximately 6 percent more of the total volume was moved into the left lane at a volume of 500 vph, decreasing to approximately 3 percent more at 1700 vph. The larger share of these vehicles came from the center lane. At less than 1300 vph flow rate, many drivers even moved left from the right shoulder lane. The point of equal lane distribution in lanes 1 and 2 was moved from the rate of 1850 vph back to approximately 1600 vph. As was mentioned earlier, this changed lane distribution moved the drivers left at this site, but they did so without increasing their speed. It resulted in a reduced mean speed for all lanes.

As normally found at a 3-lane site of low volume, the center lane carries a larger portion of traffic than the left lane. In this case, at a volume of 600 vph, there were approximately 20 percent of the vehicles in lane 1 and 50 percent of the vehicles in lane 2 in the "before" condition. This is evidence of the fact that many drivers do consider the left lane for passing only when volume is low. But, with a shift to approximately 25 percent and 45 percent respectively in lanes 1 and 2 at the same volume, we would be discouraging the attitude of keep right and pass left.

For the 3-lane site on the Bayshore Freeway, Figure 9 again shows results of a traffic shift to the left lane, although of a lesser magnitude. Even at these higher flow rates, the signs did affect the lane distribution. These flow rates, however, are still well below capacity.

The Harbor Freeway site, with volume distribution results shown in Figure 10, exhibits little significant change. The "after" condition does, however, tend to group all lane distributions a little closer about the 25 percent per lane range. As the number of lanes increases, and as volume increases, the driver has less tendency to consider the left lanes as passing lanes, and also has less of an opportunity to use them for this purpose.

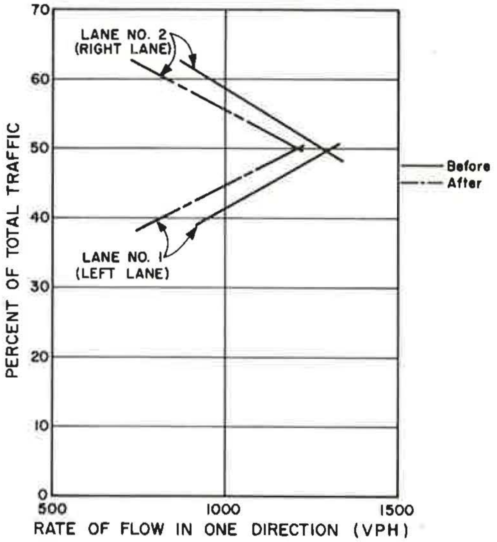

Figure 7. Effect of minimum speed signs on volume distribution by lane at 4-lane site—I-80 near Roseville.

MANEUVERS

Lane-changing and passing maneuvers were observed by two different methods and at two locations within each study site. At the leading set of signs at several sites, these maneuvers were visually observed and recorded between limits of ¼ mile upstream of the sign and ¼ mile downstream of the sign. The purpose was to determine whether or not the signing had any effect on driver behavior when he first observes the signs. Another observation was then made downstream, near the end of each study site. This observation was made within limits of approximately ¼ mile, using time-lapse photography, and its purpose was to determine what effect continuous minimum speeds by lane would have on lane-changing and passing maneuvers.

Lane Changing

The observation of lane-changing maneuvers at the leading set of signs was conclusive. There was a 38 percent increase in this maneuver in the "after" condition. The

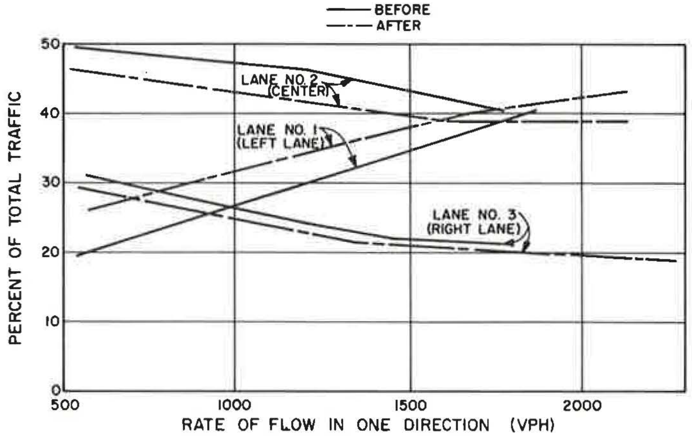

Figure 8. Effect of minimum speed signs on volume distribution by lane at 6-lane rural site—I-80 at Dixon.

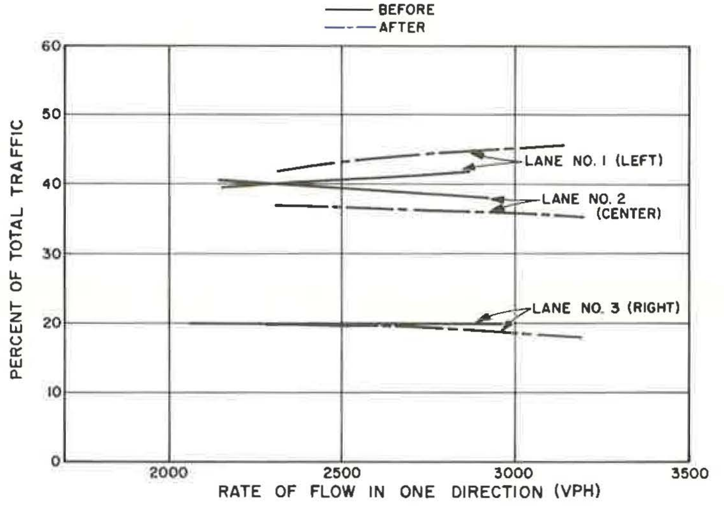

Figure 9. Effect of minimum speed signs on volume distribution by lane at 6-lane metropolitan site—Bayshore Freeway at Ralston Avenue.

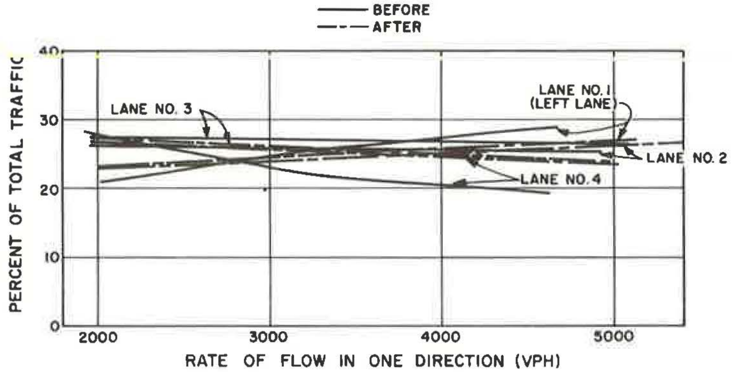

Figure 10. Effect of minimum speed signs on volume distribution by lane at 8-lane site—Harbor Freeway at 149th Street.

increase was realized in all lanes. This would indicate that some drivers felt a need for changing lanes when observing the signs. From the results of the volume distribution, as discussed earlier, it is now realized that the signs urged drivers to move to the left lanes. Where traffic was very light, they stayed in the left lane and did not discourage others from using that lane because of little or no conflict. However, when traffic volumes were moderate to heavy, there is an indication that some of the drivers who had moved left, returned to the right or caused other drivers to move right before reaching the data collection location, approximately 4 miles downstream.

Table 10 shows the results of the downstream lane-changing observation. The "before" and "after" observation periods and time of day were identical and included not less than 3 hours each. Any reduction of lane-changing in the "after" condition is considered a positive factor in the interest of safety. It may be observed that at the Roseville and Dixon locations there is evidence of more lane-changing in most lanes. There is no significant reduction. At the 3-lane Bayshore Freeway study site (higher volume), lane-changing decreased in all lanes. At the Harbor Freeway (also high volume), lane-changing decreased for vehicles moving to the right and showed some increase for vehicles moving to the left lanes.

TABLE 10

OBSERVED LANE CHANGES PER ¼ MILE NEAR DOWNSTREAM SIGNS

Location	Before						After					
	1-2*	2-3	3-4	4-3	3-2	2-1	1-2	2-3	3-4	4-3	3-2	2-1
I-80 at Spruce Avenue	169	–	–	–	–	201	225	–	–	–	–	201
I-80 at Dixon Road (Weekday)	16	11	–	–	7	13	13	12	–	–	9	12
I-80 at Dixon Road (Sunday)	94	49	–	–	56	69	94	62	–	–	73	96
Bayshore Freeway at Ralston Avenue	160	160	–	–	77	104	110	135	–	–	67	64
Harbor Freeway at 149th Street	80	132	98	83	66	34	52	91	93	103	65	43

* Notation indicates lane numbers changed from and to; for instance, 1-2 indicates changes from lane 1 (adjacent to median) to lane 2.

Passing on the Right

The observation of passing at the leading set of signs showed increased passing on the right of all left lanes. There was 77 percent more passing on the right in the "after" observation. This would indicate that as some drivers changed lanes by generally moving to the left, they did not increase their speed. They were, therefore, passed on the right.

TABLE 11

PASSES ON THE RIGHT OBSERVED PER ¼ MILE NEAR DOWNSTREAM SIGNS

Location	Before			After		
	2-1*	3-2	4-3	2-1	3-2	4-3
I-80 at Spruce Avenue	22	–	–	10	–	–
I-80 at Dixon Road (Weekday)	9	3	–	8	10	–
I-80 at Dixon Road (Sunday)	31	45	–	47	56	–
Bayshore Freeway at Ralston Avenue	25	25	–	23	25	–
Harbor Freeway at 149th Street	31	30	32	38	61	38

* Notation indicates vehicle in lane numbered passing vehicle in adjacent lane numbered; for instance, vehicle in lane 2 passed vehicle in lane 1, shown as 2-1.

The results of the passing on the right observation at the downstream location of each study site are given in Table 11. While Table 11 shows that passing on the right increased, the numbers involved are relatively small; e.g., at the Dixon site, during a 5-hour Sunday period before signs were erected, 76 passings on the right occurred, and during the after period, 103 passings on the right occurred. Neither of these numbers is very important when it is remembered that thousands of vehicles went by during these periods.

ENFORCEMENT

Insight into enforcement problems was gained primarily from incident interviews with violators of the minimum posted speeds. Many comments were also obtained from discussions with law enforcement officers. All incident reports and comments pointed to one definite conclusion: Minimum speed limits are difficult to enforce.

Probably the primary reason for this difficulty of enforcement is speedometer error. Automobile speedometers generally read higher than actual speed. Even tire wear may make a difference of 3 or 4 percent. The error in some older car speedometers is extremely high, and many speedometers do not even function. From the incident reports, it was learned that 29 percent of the violators indicated they thought they were traveling at a speed greater than the posted minimum.

As posted, for purposes of this study, we were asking the motorist to drive at a minimum speed of only 5 mph less than the maximum allowable (most lanes). Yet, it was observed that the driver traveling 60 mph in the extreme left lane was often impeding traffic under the vehicle code. The posted minimum would make it difficult to cite these violators.

SIGNING

The signs used for this study were regulatory (white letters on a black background). The sign over each lane of traffic was 9 ft wide and 7 ft high. The sign messages were all similar to those shown in Figure 2.

A significant problem in minimum speed signing lies in the word "minimum" itself. The word "minimum" was frequently read as "maximum," if we may believe the claims of drivers stopped by patrolmen for violating the minimums—13 percent of the violation incident reports indicated this fact. It would, therefore, be necessary to devise a word message which excludes the word "minimum" if the signing is to be most effective. Another way to avoid this confusion might be to show maximum and minimum speeds on the same sign.

Another significant problem is one common to all signing. Many signs are not seen by the motoring public. Approximately 24 percent of the minimum speed violators indicated that they had not seen the signing. This rate was evidenced by the incident reports obtained. The problem of communication with the driver has been a matter of increasing concern to traffic engineers. Freeway driving, particularly in metropolitan areas, is quite demanding and signs are often missed. One reason, of course, is the existing profusion of signs. Adding still more signs with multiple messages (one for each lane) has obvious drawbacks in this respect, as well as aesthetically. Methods of communicating with drivers by means other than visual have been under consideration for several years, but no practical method has yet been developed.

For this study, the minimum speed signs were all placed on overcrossings. The availability of overcrossings, however, is quite limited. Many of the newer freeways in metropolitan areas have been built on embankment with undercrossings in lieu of overcrossings. Many of the existing overcrossings are already being used for other signing. It would, therefore, be necessary to construct sign bridges. If the minimum-speed sign spacing was set at 2 miles, the cost of the sign bridge installation would be approximately \$7,000 per mile for a 4-lane facility, \$8,200 per mile for a 6-lane facility and \$10,000 per mile for an 8-lane facility. There are approximately 2,000 miles of existing freeway in California. The annual maintenance cost would be in addition to this installation.

The installation of sign bridges on a broad basis would significantly increase the exposure of the motorist to fixed objects. Fixed objects constitute 25 percent of all freeway accidents, and 31 percent of freeway fatal accidents (2). Fixed-object accidents have a much higher fatality incidence than other accidents.

A recent study (3) showed a high severity rate for fixed-object accidents, and recommended removal of any unnecessary signing. It is therefore realized that we would decrease the overall safety of freeways by increasing the number of signposts.

CONCLUSIONS

Minimum speeds by lane cannot be considered as a relief for traffic congestion. Congestion occurs when a portion of the roadway reaches capacity. As traffic volumes approach capacity, drivers are forced to reduce their speeds. The mean speed on a mainline roadway at capacity is approximately 35 to 45 mph. Minimum speed signing, therefore, would have no effect when a roadway is operating at capacity.

The desired results of minimum speeds by lane would be (a) decreased travel time for the fast driver, (b) less frustration to a driver being delayed, and (c) increased safety. All of these desired results could only hope to be achieved when traffic volumes are well below capacity. From the results of this study, none of the desires were realized at any volume.

Probably the most unanticipated result of this study was the fact that the minimum-speed signing generally moved more drivers into the left lanes instead of moving slow drivers to the right lane. This was contrary to the intent of the signs, i.e., that drivers should keep right and pass left. This shift generally caused (a) a reduction of mean speeds for vehicles traveling in the left lanes, (b) increased passing on the right, and (c) increased, rather than reduced, travel time.

From an operational point of view, it may be definitely concluded that imposing minimum speeds by lane showed little or no positive advantages and showed some definite disadvantages.

The study also pointed to the fact that speeds in the left lanes of traffic are very close to the legal maximum speed limit. A vehicle traveling at the minimum posted speed in the left lanes would often be impeding other traffic because usually more than 95 percent of the traffic in that lane is traveling faster than 60 mph. This would lead to the conclusion that, to be effective, minimum speed limits should be even higher than those used and perhaps the minimum speed limit should be even higher than the maximum speed limit. This is not feasible, however, with speedometer error, safety considerations, and the like.

The minimum speed signs were posted for only 2 to 4 weeks at each study site. This, of course, was too short a period to evaluate the accident picture with a before-and-after comparison. There are, however, several conclusions to be drawn regarding safety. This study has provided no evidence that a freeway posted with minimum speed signs would induce a safer operation. There was no decrease in short headways (tailgating). There is, however, evidence that the overall safety of the freeway would be less, due to the installation of fixed objects (sign bridges).

In summary, it should be concluded that minimum speed by lane signs would only add clutter to the highways, with definite operational and safety disadvantages.

ACKNOWLEDGMENTS

The work reported was done for the California Division of Highways by the Traffic Department. The study was under the general direction of Karl Moskowitz and E. F. Graham. The co-investigator and designer of the study was Roger T. Johnson.

The author wishes to acknowledge the assistance of the Traffic and Maintenance Departments of Districts 03 (Marysville), 04 (San Francisco), 07 (Los Angeles), and 10 (Stockton) for providing the help with sign erection and data collection for this study. Recognition is also given to the Materials and Research Department (Sacramento) for providing the time-lapse photography and other technical assistance. The cooperation and assistance of the California Highway Patrol and the Los Angeles Police Department is greatly appreciated.

This project was accomplished in cooperation with the U.S. Bureau of Public Roads. The opinions, findings, and conclusions expressed are those of the Division of Highways and not necessarily those of the Bureau of Public Roads.

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1. Taragin, A., and Hopkins, R. C. Traffic Analyzer: Its Development and Application. Proc. Inst. Traffic Engineers, p. 263, 1960.
2. Johnson, Roger. Freeway Fatal Accidents—1961 and 1962. California Division of Highways, p. 6-9, Nov. 1963.
3. Objective Criteria for Guardrail Installation. California Division of Highways, July 1966.

Appendix

SPEED DISTRIBUTION CURVES
(Figures 11 through 25 Shown on the Following Pages)

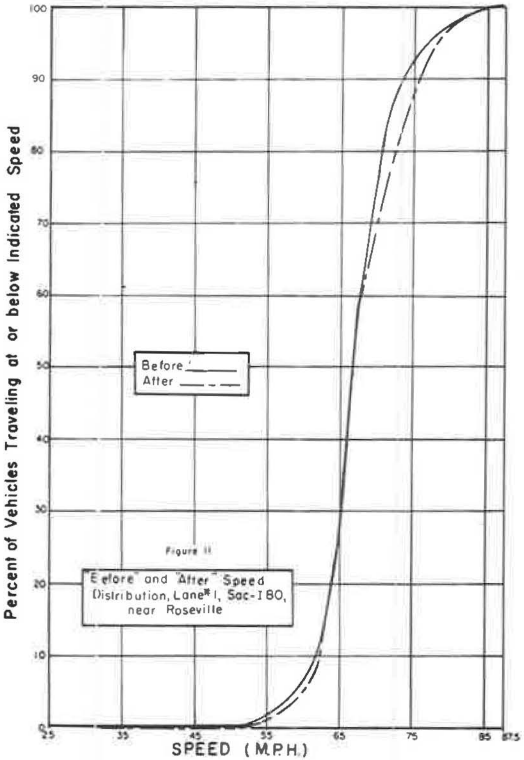

Figure 11
"Before" and "After" Speed Distribution, Lane #1, Sac-I 80, near Roseville

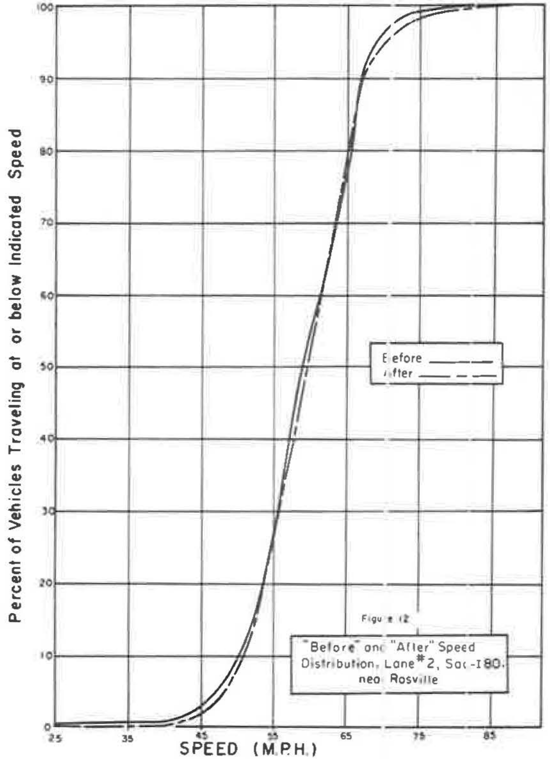

Figure 12
"Before" and "After" Speed Distribution, Lane #2, Sac-I 80, near Rosville

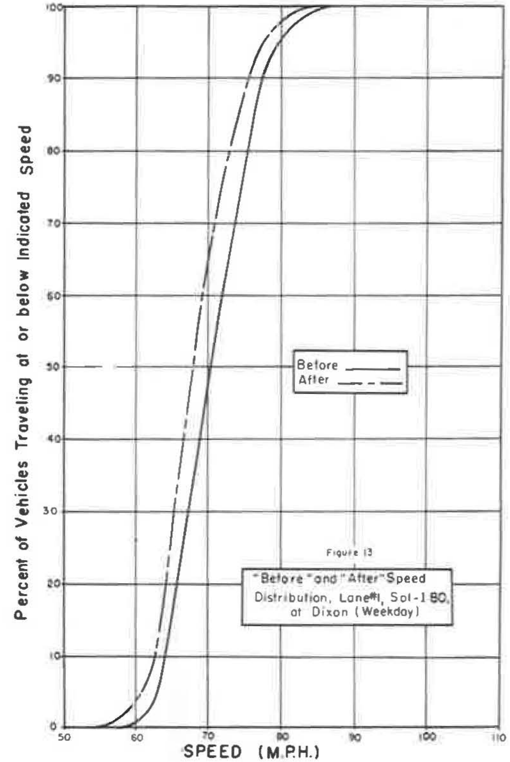

Figure 13
"Before" and "After" Speed Distribution, Lane #1, Sol-I 80, at Dixon (Weekday)

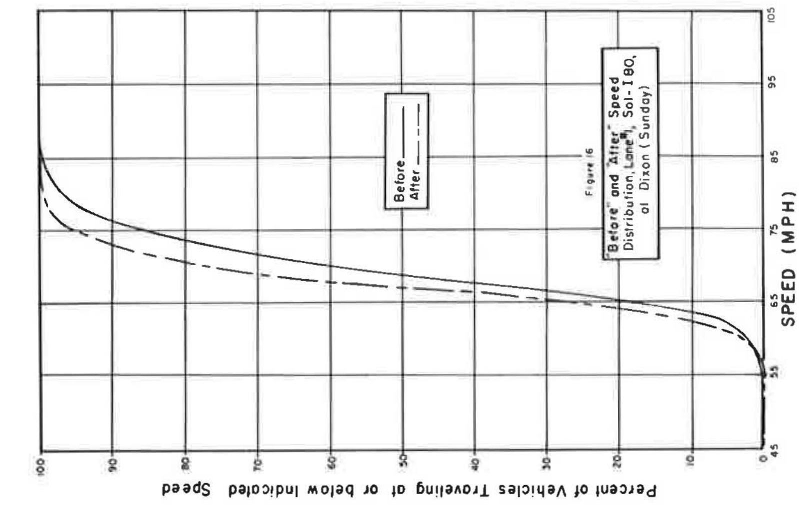
Figure 16
"Before" and "After" Speed Distribution, Lane#1, Sol-I 80, at Dixon (Sunday)
Before
After
SPEED (MPH)
Percent of Vehicles Traveling at or below Indicated Speed

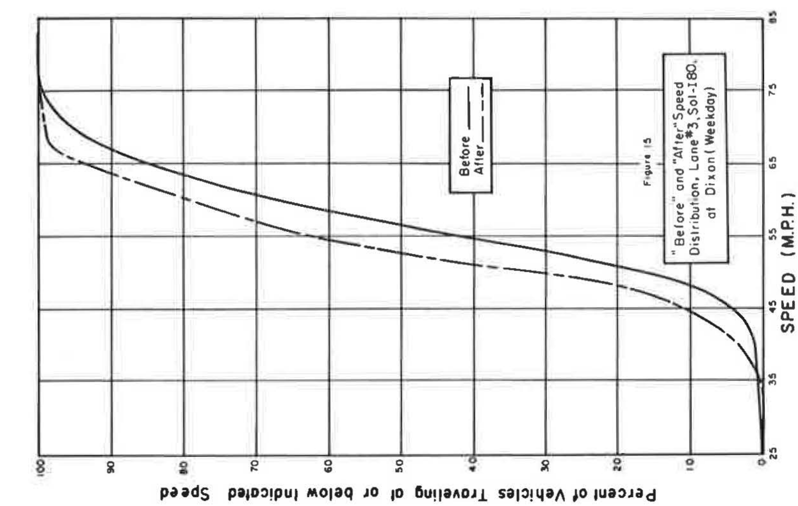
Figure 15
"Before" and "After" Speed Distribution, Lane#3, Sol-I80, at Dixon (Weekday)
Before
After
SPEED (M.P.H.)
Percent of Vehicles Traveling at or below Indicated Speed

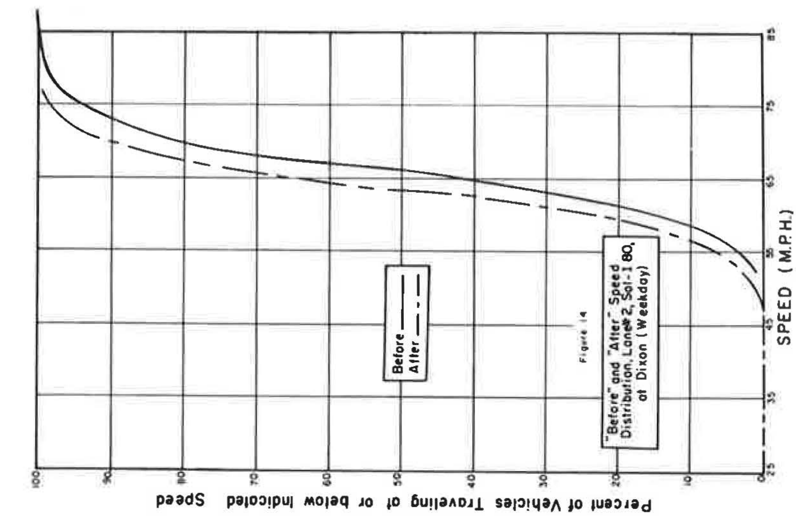
Figure 14
"Before" and "After" Speed Distribution, Lane#2, Sol-I 80, at Dixon (Weekday)
Before
After
SPEED (M.P.H.)
Percent of Vehicles Traveling at or below Indicated Speed

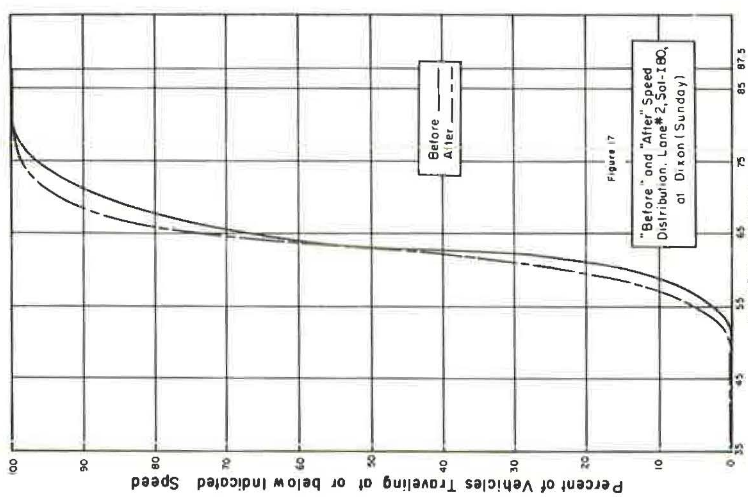

Figure 17

"Before" and "After" Speed Distribution, Lane# 2, Sol-I 80, at Dixon (Sunday)

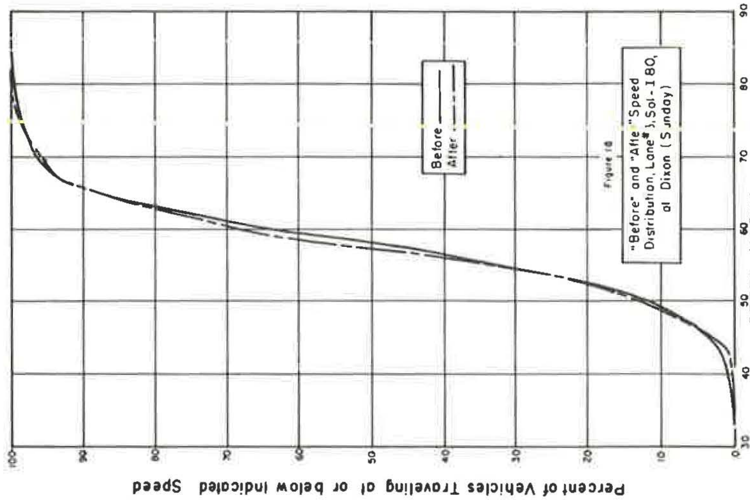

Figure 18

"Before" and "After" Speed Distribution, Lane# 1, Sol-I 80, at Dixon (Sunday)

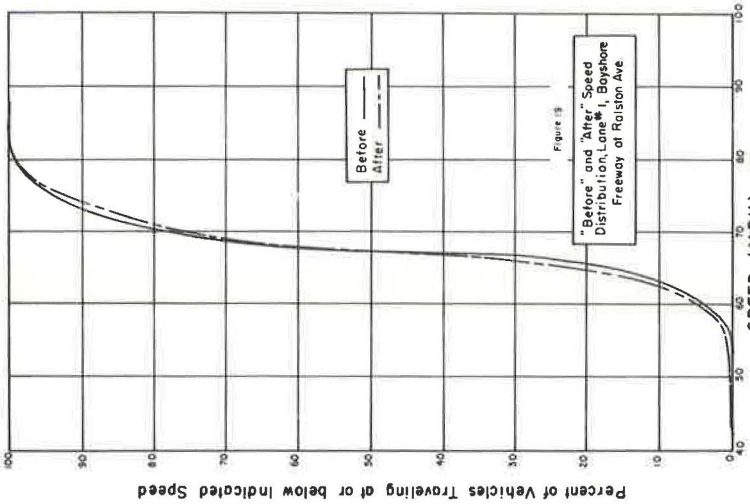

Figure 19

"Before" and "After" Speed Distribution, Lane# 1, Bayshore Freeway at Ralston Ave.

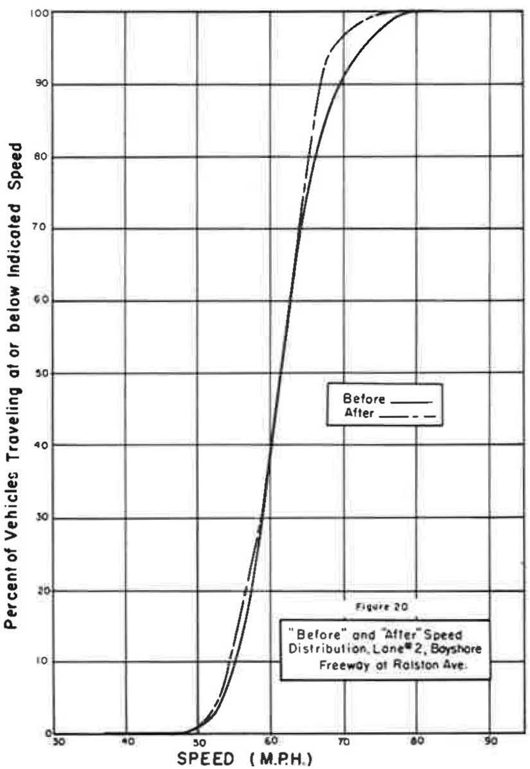

Figure 20

"Before" and "After" Speed Distribution, Lane # 2, Bayshore Freeway at Ralston Ave.

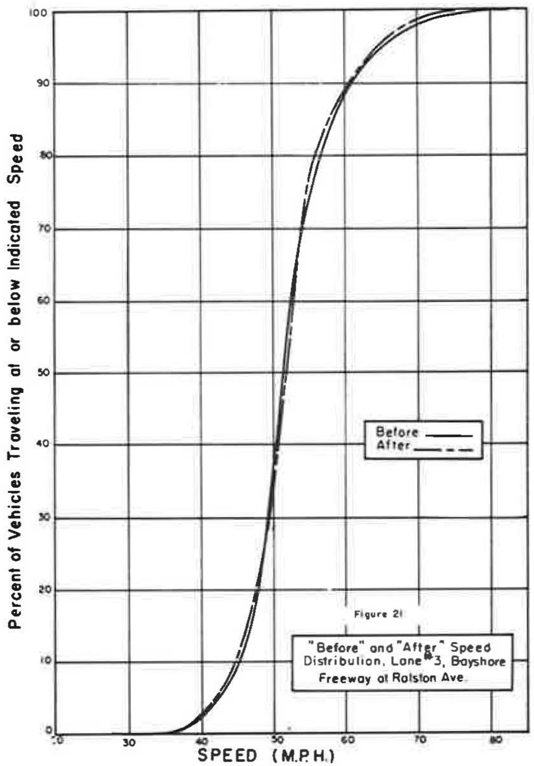

Figure 21

"Before" and "After" Speed Distribution, Lane # 3, Bayshore Freeway at Ralston Ave.

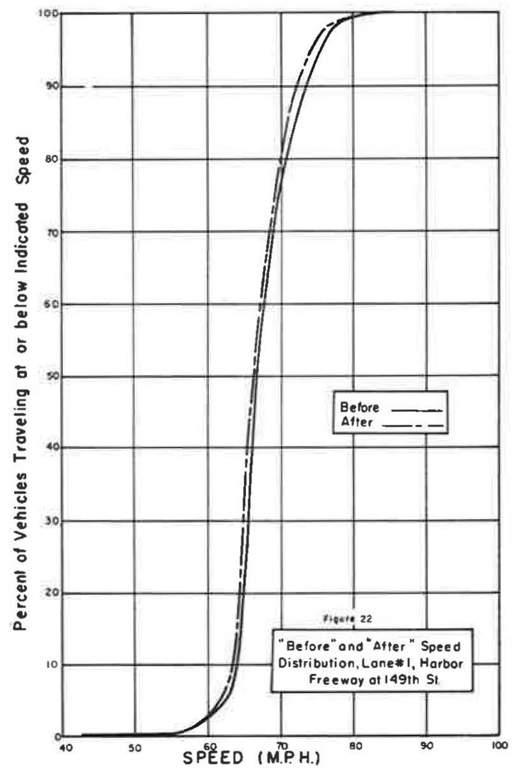

Figure 22

"Before" and "After" Speed Distribution, Lane # 1, Harbor Freeway at 149th St.

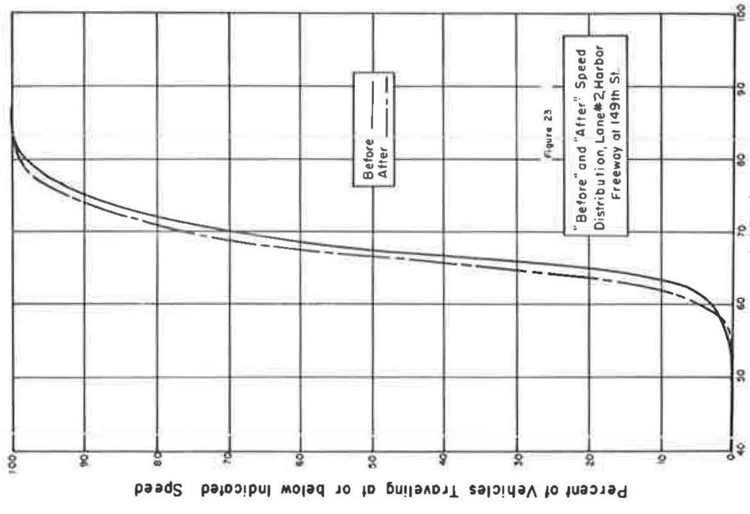

Figure 23 "Before" and "After" Speed Distribution, Lane#2, Harbor Freeway at 149th St.

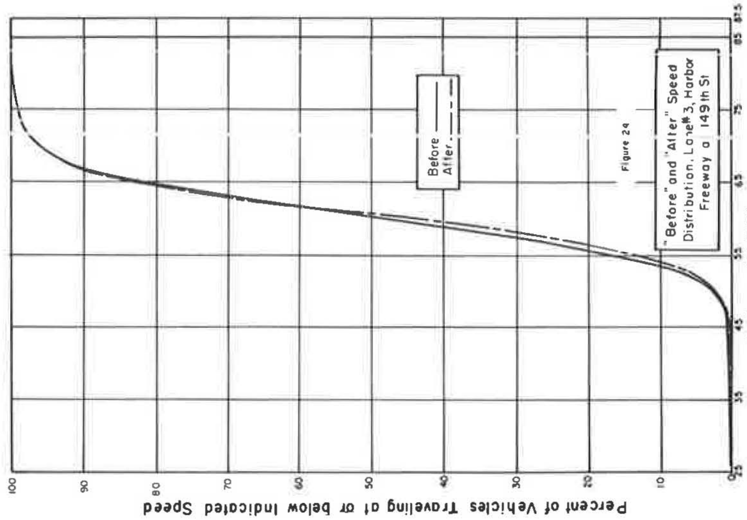

Figure 24 "Before" and "After" Speed Distribution. Lane#3, Harbor Freeway a 149th St

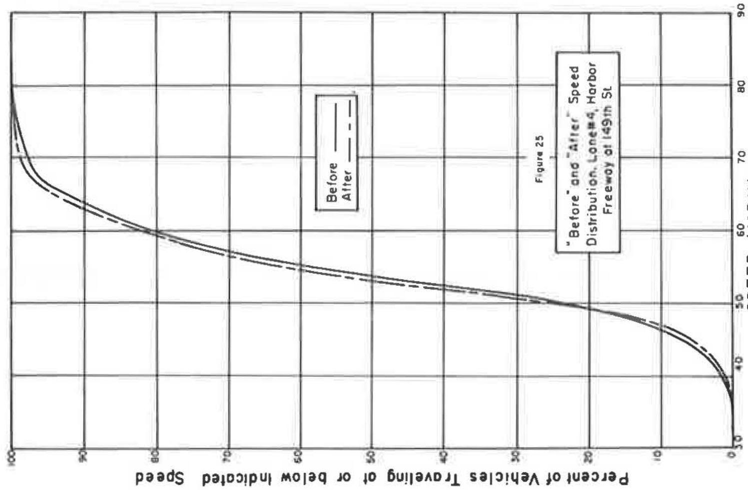

Figure 25 "Before" and "After" Speed Distribution, Lane#4, Harbor Freeway at 149th St

Discussion

JOHN J. HAYNES, Professor and Head of Civil Engineering Department, University of Texas at Arlington—This interesting research project is important in contributing new knowledge concerning the operation of vehicles on freeways. The conclusions seem to be substantially valid. This study and its associated research have shed new light on a very real problem. The concept of directing the drivers in each lane, by means of proper signing, to travel within a certain small range of speeds would at first seem to be a proper approach to the operation of vehicles on freeways. It is, in fact, an extension or refinement of the old rule of traffic which asks that "slow drivers keep right." As the author indicated, this particular approach is aimed primarily at the operation of traffic on multilane facilities having low densities and high levels of service. It is not likely that this approach would be of any value except in levels of service A or B. A question raised early in this research project might be: "Since freeways with low volumes of traffic operating at low densities present few conflicts and only minor operational problems, why should an attempt be made to improve on a condition that is admittedly very satisfactory?"

This report indicates that a majority of vehicles traveling in the left lane of these freeway sections were speeding; that is, they were traveling in excess of 65 mph. Indeed, the data indicated that the average speed of this left lane was in excess of 67 mph. One of the stated objectives of the research was to determine whether the average speed in each of the lanes could be increased. Such an increase, it was stated, would be deemed an improvement. It would seem questionable that an increase in the number of speeders should be called a figure of merit. It rather might be called a detriment to the operation of the freeway.

It was reported that one of the unexpected results of this signing was more shifting to the left and less shifting to the right. A major reason many individuals moved left because of this signing is, undoubtedly, attributable to the inaccuracy of their speedometers. The authors have rightly pointed this out. Only a small percentage of speedometers are accurate to within 5 percent and many new automobiles are sold with speedometers in error over 15 percent. (I happen to own one.) Usually the error is on the safe side; that is, the speedometer reads higher than the true speed. The total range of speeds required in the left, or median lane, by the signing was only about 8 percent of the average lane speed. It is likely that inaccurate speedometers were the principal reason that this particular study showed no beneficial effects. Calibrated speedometers are available on automobiles at some extra cost. It might be well to recommend that all speedometers be made accurate to within some small percentage of error, say plus or minus 2 percent for the particular rear-end ratio and rear tire size of the vehicle. This seems a reasonable recommendation in the light of many other restrictions now being imposed upon the automobile manufacturers.

Since the study sites were comparatively short and had only a few sets of signs, the question arises whether the motorists really knew when this particular speed requirement had ended. Furthermore, since the signs had been in place at least 2 weeks prior to taking any data, it could further be questioned whether the people who regularly used the facility had a knowledge that it was required for only a few miles and thus were less responsive by the time the data were taken.

In the determination of mean speeds and deviations, it was stated that the volumes were well below capacity when the data were collected. The maximum volumes were approximately 1300 vph per lane and the minimum volumes were only about 200 vph per lane. The volumes on each particular study site varied from some minimum value to about double the minimum value. There is a considerable difference in the speeds to be expected at volumes of 1300 vph per lane and volumes of 700 vph per lane. The speed data were not related to volume data well enough to permit a detailed study of the speed-volume relationships. The minor differences in means and standard deviations that resulted from the analysis of the before-and-after data become less meaningful since the effects of volume on the speed data were not considered.

The overall travel times alluded to in the report were apparently determined by extrapolating spot speed information for the vehicles observed. It should be pointed out again that the very small increases or decreases in travel time would be more a function of the volumes existing at the time the data were taken than indicative of the result of the signing alone. Another point to be made in connection with these small calculated increases in overall travel time is that the drivers might not have been able to have detected the increase. It could have been interesting if the motorists could have been polled to see if they thought they were traveling slower or if they thought they were traveling a little more quickly. They, in fact, might have indicated that they considered the signing better for them and that they believed their travel time was less. As a research project conducted in response to a legislative request, it could have been interesting to determine what the motorists thought of the signing.

In the study of short headways, the data points shown in Figures 3, 4, 5, and 6 indicate that there was a broad range or grouping of volumes for the purposes of relating short headways to volumes. Volume groups with ranges of about 15 percent seem to be too broad to yield sufficiently accurate headway distribution data. Headways, or the time interval between the passage of successive vehicles, actually form the basis of the rate of flow. Each headway, then, is indicative of a flow rate, and thus flow rates can be exactly as variable as headways are. Because of the close interrelationship between headways and flow rates, the analysis of short headways should be carefully done within small intervals of volume. By using the given lane distribution, this discussant calculated the expected percentage of small headways according to a Poisson, or random, distribution for several total volumes on a few of the study sites. The observed percentages of small headways were somewhat less than the expected percentages for both the "before" and "after" studies.

In their discussion of volume distribution by lane, the authors have included some figures (Figs. 7, 8, 9, and 10) without showing any data points. Such data points would have been instructive. Each lane distribution-volume relationship was represented essentially by one or two straight-line segments. This relationship can usually be represented more closely by a second-degree parabola.

Concerning the subject of lane changing, it was pointed out that there was a significant increase in this particular maneuver in the "after" condition, and that "it is now realized that the signs," in fact, "urged drivers to move to the left lanes." It was stated that when traffic was very light, drivers stayed in the left lane and that this did not discourage others from using that lane because little or no conflict existed. However, when traffic volumes were moderate to heavy there was an indication that some of the drivers who had moved left returned to the right or caused other drivers to move right before reaching the data collection location. A description of the volumes observed should not include the term "heavy," because only light to moderate volumes were observed in this research. It is also stated that the "before" and "after" periods of observation during the day were identical and included at least 3 hours each. Volume changes occurred within these 3-hour periods. It is unfortunate that the relationship of lane changing vs volumes was not included in the analysis; such data could be valuable.

The author indicates, in connection with the discussion of the signs themselves, that a significant problem was that the signs were apparently not seen by many motorists. This is a problem associated with all types of signing. It was stated that approximately 24 percent of the minimum speed violators indicated they had not seen the signing. It would have been of value to have determined how many times these persons had traveled the study site. It would not be too surprising to find that a driver had not noticed the signs if it was the first time he had traveled the study site. It would be quite discouraging, however, to find that an individual who had traveled the route regularly for over 2 weeks had never noticed the signs.

The cost of such signing was estimated at $7,000 to $10,000 per mile of freeway. Certainly, it would be agreed that the results of this research would not indicate such expenditures would be justified. Furthermore, as indicated, the required sign standard would present additional fixed objects within the right-of-way and thus create additional safety hazards. The study actually produced no evidence of operational advantage due to minimum speed signing by lane.

The author is to be commended for a very interesting report. This work will be of interest to many who are involved in the operation of freeways.

J. L. VARDON, Metropolitan Toronto Planning Board—There is little doubt that the author has presented a comprehensive and complete report of findings with respect to minimum speed limits by lane on freeways. It is a fine piece of work and should become a prime reference on the subject for some time to come.

It is surprising to realize that a major effort on this topic has been so long forthcoming in light of the many suggestions for minimum freeway speed limits in the past 10 years or so. The investigation is timely and worthwhile for many road authorities in North America.

The justification for minimum speed limits and the source of the suggestions would make an interesting study in itself. The source of the request in this case is typical, wherein the California Legislature asked for a study to determine if slow speeds on any part of a state highway consistently impede the normal and reasonable movement of traffic. No evidence was presented to indicate that there was a problem and that there was sufficient justification for increased control.

Isolated occurrences of problems and poor driver habits tend to create an undue impact on the observer. The study was designed to investigate the traffic characteristics of the total driving universe, by lane, for the specific locations. It was not designed to place any particular emphasis on the observation of "rare events". Hence the lack of spectacular effects of the proposed control device could be anticipated.

Fortunately, the author has been able to translate the mandate into five reasonable criteria. However, it is significant to note that the underlying premise has been one of free-flow or quasi-free-flow conditions. Hence the investigation becomes a study to determine the effect of minimum speed limits by lane on free-flow conditions. This makes infinitely more sense, as pointed out, than a determination of minimum speed limits under congested conditions. Nevertheless, there are some difficulties with the former case because of the relation between the existing maximum limit and the assumed minimum limit.

The maximum speed limit on the majority of California freeways is 65 mph. The average speed for the left lane of these freeways is "in the range of 67 mph". The minimum speed limit for the left lane was set at 60 mph. Hence the range of available legal speeds is very small and, possibly, not within the accuracy range of the average speedometer nor within the capability of the average driver to maintain his speed in this narrow range.

In light of the foregoing plus the relatively low minimum speed violation rate, one wonders about the regard that any driver in the left lane has for a speed limit—particularly a minimum. Since the average left lane speed is in excess of the maximum speed limit, it is suggested that the higher legal limit would be of more concern for a majority of the drivers in the left lane. The slight decrease in observed "after" speeds could be explained by a keener driver awareness of increased speed control, surveillance and/or enforcement.

If material benefits were to accrue from a minimum speed limit, it is suggested that they would manifest themselves most strongly in lane 2 of a 3-lane roadway and lane 3 of a 4-lane roadway. The higher maximum-minimum speed differential plus the lower absolute speeds gives the driver a better opportunity to comply and maneuver within the intent of the speed control device. The right-hand lane is likely not typical because of the presence of trucks. However, the data present little substantiation for such a subtle hypothesis. Certainly the speed and standard deviation data do not reinforce it.

The increased minimum speed violation rate for these lanes might be construed to mean that drivers were attempting to comply with the minimum limit but that the error in their speedometers gave them false speed information and hence put them below the 55-mph limit. Here too there is little credence to the suggestion because there is no confirmation by the speed date, a reduced number of lane changes, nor a reduction in the right-hand passing.

The author has stated that "Headways between vehicles are tied to traffic volume by mathematical laws." He then presents an arithmetical example involving the relation between the gaps created by a hypothetical volume and the number of seconds in one hour. In this sense he is correct but the basic statement is misleading. Headways between vehicles are not governed by mathematical laws, unfortunately, but can be shown to approximate known theoretical mathematical distributions.

The results of this investigation have brought into focus the main considerations and problems of minimum speed limits by lane under free-flow conditions. In spite of the low differential between maximum and minimum limits, one would tend to conclude that materially improved vehicular control could not be achieved by this technique. The paper does imply some important questions with respect to the necessity for increased freeway controls, the conditions that should dictate their use, and the techniques to be employed that will give the desired result. One wonders whether a prima facie speed control or other legislative action would not be more appropriate for any problems associated with the infrequent slow driver.

T. DARCY SULLIVAN, Senior Traffic Field Engineer, Illinois Division of Highways—Regulation of vehicular speeds is one of the most controversial and important problems in traffic operations today. It is controversial because of the wide differences of opinion that exist among engineers, enforcement officers, motorists, legislators, and the general public concerning the solution of a speed problem. It is important because the severity of accidents that occur at high speeds is much greater than for those occurring at low speeds.

The need in speed regulation is for speed controls that are realistic, can be easily and impartially enforced, and enhance smooth traffic flow on the roadway. By realistic, I mean that the control should be readily accepted by the motorist and thus, to a great extent, be self-enforcing. Easily enforceable means that undue burden has not been placed on the arresting officer by making the conditions of the arrest so numerous that the case is difficult to substantiate in court. Traffic flow may be enhanced through modification of drivers' habits as measured by speed, volume, density, and other traffic flow characteristics. Although these needs are generally thought of as applying to maximum speed limits, there is no reason to think that they cannot be just as validly applied to minimum speed limits.

How does the concept of assignment of minimum speed limits by lane satisfy the needs of realism, enforceability, and smooth traffic flow? The concept seems to be a realistic one in terms of driver acceptance. The system evaluated in the project under discussion or a modification of it has been proposed in many different areas. In fact, it had sufficient popular support to be brought before the California Legislature in the form of a resolution. Another indication that it was accepted by the public was the fact that, according to the research results, generally less than 5 percent of the motorists violated the minimum speed restriction.

From an enforcement point of view, the system certainly presents problems. For the usual minimum speed violation summons, the arresting officer only has to prove that the offending motorist was traveling below a specified speed. The officer has only to make the necessary allowance for equipment and speedometer errors and he can prove his case beyond a reasonable doubt. Under the proposed system, the officer would have to prove that the motorist was driving in the specified lane and was not slowing down in an attempt to move into a lane farther to the right. Either of these items might be used as a defense by the motorist thereby casting doubt on the validity of the summons. Discussions with several enforcement officials have indicated that these factors would make the proposed system extremely difficult if not possible to enforce.

The author has defined four measurable traffic flow characteristics that he believed might be indicators of reduced internal traffic stream conflicts. These included a more uniform speed distribution, fewer platoons of vehicles, longer headways, a decreased number of lane changes, and a reduced number of vehicles passing on the right. These criteria are well chosen and express most, if not all, of the desirable effects minimum speed regulation might have. Unfortunately, the posting of minimum speed limits by lane had little influence on some of the variables and negatively influenced the results on most of the others.

Based on the previous discussion, it appears that the posting of minimum speed limits by lane satisfies only one of the three basic needs in a good speed regulation program: the need of realism. If we then reject the concept of minimum speed limits by lane, what alternatives do we have to the solution of the basic problem of vehicles driving in the left lane or lanes and blocking other traffic that desires to travel at a higher rate of speed?

One of the most obvious alternatives is the "Keep Right Except to Pass" sign. This sign is used in many states where the state law requires vehicles to stay in the right lane at all times except when overtaking and passing. The regulation has been readily accepted by most motorists in the states where it has been used and is easily enforced. Unfortunately, while the sign may produce entirely satisfactory results on very light volume roadways, on more heavily traveled roads it results in an excessive number of lane changes. A vehicle traveling at a slightly higher than average rate of speed has a choice of staying in the left lane or returning to the right lane between each vehicle that is overtaken and passed. A driver, choosing the second alternative in compliance with the law, thus becomes involved in almost continuous lane-changing maneuvers. The larger number of lane changes thus generated not only produces a potentially hazardous situation, but it may also reduce the capacity of the highway and the level of service being provided. The use of this sign would meet two of the needs previously established: realism and enforceability.

Another sign that may be used is the "Slower Traffic Keep Right" sign. This sign is being used increasingly in many states and has been readily accepted by the motoring public. Discussions with enforcement officials indicate that this sign is just as easily enforced as the "Keep Right Except to Pass" sign under most conditions. With this sign posted, a motorist would be permitted to drive in any lane so long as he was not overtaken by another motorist who desired to pass. However, as soon as he was overtaken, he would be under an obligation to move to the right to make room for the faster vehicle. This sign would encourage motorists to drive in the right-hand lane but allow them to remain in the left lane so long as they did not impede other traffic. The number of lane change maneuvers would thereby be minimized. This sign appears to meet all three needs of realism, enforceability, and enhancement of smooth traffic flow.

An additional technique, which has been used on the Chicago Metropolitan Area expressways for several years, is the restriction of trucks to the two right lanes except in the vicinity of left-hand entrance or exit ramps. This regulation has been well accepted by truckers in the area. A law recently passed by the Connecticut General Assembly provides for similar restrictions on all freeways in that state, upon posting of signs by the State Highway Department. The City of Chicago Police Department and the Illinois State Police, both of whom patrol the freeway system in the Chicago Metropolitan Area, have encountered few problems in enforcing the "Trucks Use Two Right Lanes" restriction. This technique automatically restricts many of the slow-moving vehicles to the right-hand portion of the roadway on facilities with three or more lanes in each direction and insures that at least one lane is available for the exclusive use of generally faster moving passenger vehicles. While this regulation meets the needs of realism and enforceability, it does not entirely satisfy the traffic flow need. Thus, it could only be considered a partial answer to the problem.

In summary, it would seem that the use of "Slower Traffic Keep Right" signs, independently or in combination with the restriction of trucks to the two right lanes, would achieve most of the traffic flow results desired, be readily accepted by the public, and not place an undue burden on the enforcement agencies. I am not implying that such regulations should be imposed on a system-wide basis. In fact, in areas where volumes are light and a level of service A or B is provided, probably no restrictions are needed. Any sign posted under these circumstances would simply be one more roadside obstacle and should not be erected unless a definite need has been established.

NORMAN C. WINGERD, Closure—The discussions presented by Messrs. Haynes, Vardon, and Sullivan have been of great value to the study of minimum speed limits. They have been stimulating and objective. Several suggestions for study of side issues have resulted.

Through this study I have been reminded of something basic to research. It is the fact that we must be realistic, ignore preconceived notions, and look for results that might not be anticipated. Prior to the collection of any data for this study, I presented the idea of minimum speed limits by lane to many of my friends and polled their opinions. Probably 80 percent of those questioned thought that implementation of the plan would yield very beneficial results. Such a plan has found favor among people in high office and even among some traffic engineers.

The basic misconception has been due to the failure to realize that where volumes are heavy, the speeds are controlled by traffic and not by speed limits.

The slow driver who impedes the flow of other vehicles is actually quite rare. We sometimes feel that their numbers are great, but it is only because each one makes us so aware of his presence.

I liked Mr. Sullivan's approach of weighing the proposed solutions relative to the problem. A solution to the impedance problem should be directed to the rare driver. With the signing of "Slower Traffic Keep Right" and a speed law that states that no person shall drive at such a slow speed as to impede or block the normal and reasonable movement of traffic, the State of California has the tools to cope with the problem through enforcement and education.

We will, however, remain vigilant for other solutions to the problem.

East Los Angeles Interchange Operation Study

ROGER T. JOHNSON and LEONARD NEWMAN, Assistant Traffic Engineers, California Division of Highways

This report describes the performance of portions of the East Los Angeles Interchange as affected by changes in striping of merging areas. The interchange is one of the busiest in California, with an average weekday volume of about 319,000 vehicles going through it. Because some of the traffic volumes are larger than the design volumes and because some of the volumes are considerably larger than when the interchange was opened to traffic, large delays have occurred regularly since completion of the interchange.

Objectives of the study are twofold: (a) to investigate the effects of merge geometrics on capacity, travel times, and delay, and (b) to test and practice procedures (aerial photography) for measuring these effects on traffic. Conclusions show that (a) channelizing freeway traffic, even with traversable pavement markings, can be extremely important, and detrimental if not done right; (b) striping that is less restrictive, and therefore more flexible in meeting variable demands, will result in less delay than striping that attempts to divide up the approach roadways in proportion to forecast overall demand; and (c) the use of aerial photography provides a practical means of evaluating the effects of geometric changes on traffic operation.

•VARIOUS studies of freeway traffic flow have shown that as volume increases speed decreases from about 60 mph at low, free-flow volumes to from 35 to 45 mph at capacity volumes. In studying the performance and service provided by a freeway, delays attributable to a volume increase are not large until the volume actually reaches capacity and vehicles begin to back up from some point on the freeway. In other words, in an urban commuter situation, if drivers are forced by sheer volume to reduce their speed to, say, 40 mph but still with smooth, uninterrupted flow, delay is tolerable and is less by an order of magnitude than it is when the arrival rate exceeds capacity (or service rate).

The major delay and concern is associated with driving at less than 35 mph, which results in stop-and-go driving. This will only occur when a section of freeway has reached capacity. The extent of the stop-and-go driving depends on the relationship of traffic demand to the capacity of the section. If demand does not quite reach or exceed capacity, then speeds will not drop below 35 mph anywhere on the freeway system. If demand exceeds capacity of a section, which then becomes a bottleneck, operation upstream of the bottleneck section will be slow and congested. The length of the back-up and the speed in the back-up depend on how much the demand exceeds capacity of the bottleneck and also the available storage space on the roadway.

The California Division of Highways, in cooperation with the Bureau of Public Roads, is studying these relationships as a part of its continuing research on traffic flow. This study describes the results of striping and channelization changes at two merges within the East Los Angeles Interchange, as well as a general description of operation and

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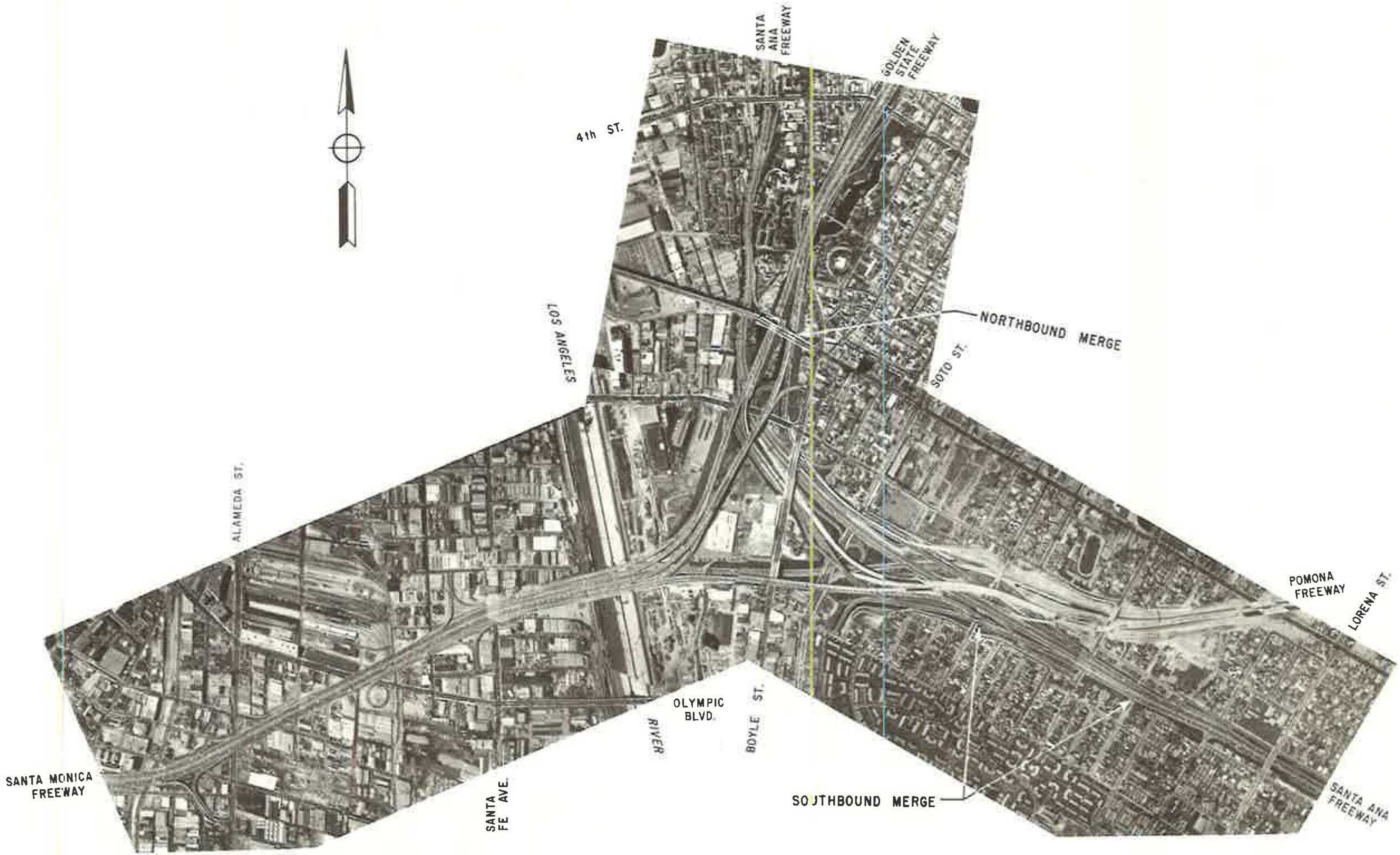


Figure 1. East Los Angeles Interchange operation study area.

effect of limited capacity. A method utilizing direct density measurements from aerial photographs was used in this study. A more detailed discussion of the methods of analysis of data from the aerial photos is presented in the Appendix. One purpose of the research is to determine the feasibility of this method for future studies of this type.

ANALYSIS

Figure 1 is an aerial view of the East Los Angeles Interchange and the study area. The East Los Angeles Interchange is the junction of the Santa Ana, Golden State, Santa Monica, and Pomona Freeways. The initial portion of the Pomona Freeway from this interchange to Third Street was completed and opened to traffic shortly after the data collection phase of this study. The study concerns operation in the interchange as affected by the northbound merge of the Santa Monica and Santa Ana Freeways to the Golden State Freeway, and also operation as affected by the southbound merge of the Santa Monica, Golden State, and Santa Ana Freeways. Figure 2 shows traffic volumes on various legs of the interchange.

Northbound Merge

In the "before" condition, the Santa Monica approach was funneled to 2 lanes, which then joined with the 2-lane Santa Ana approach to form the 4-lane northbound Golden State Freeway, as shown in Figure 3.

Conditions were such that during peak periods there would be long back-ups on the Santa Monica leg of the interchange and well back into the Santa Monica Freeway itself.

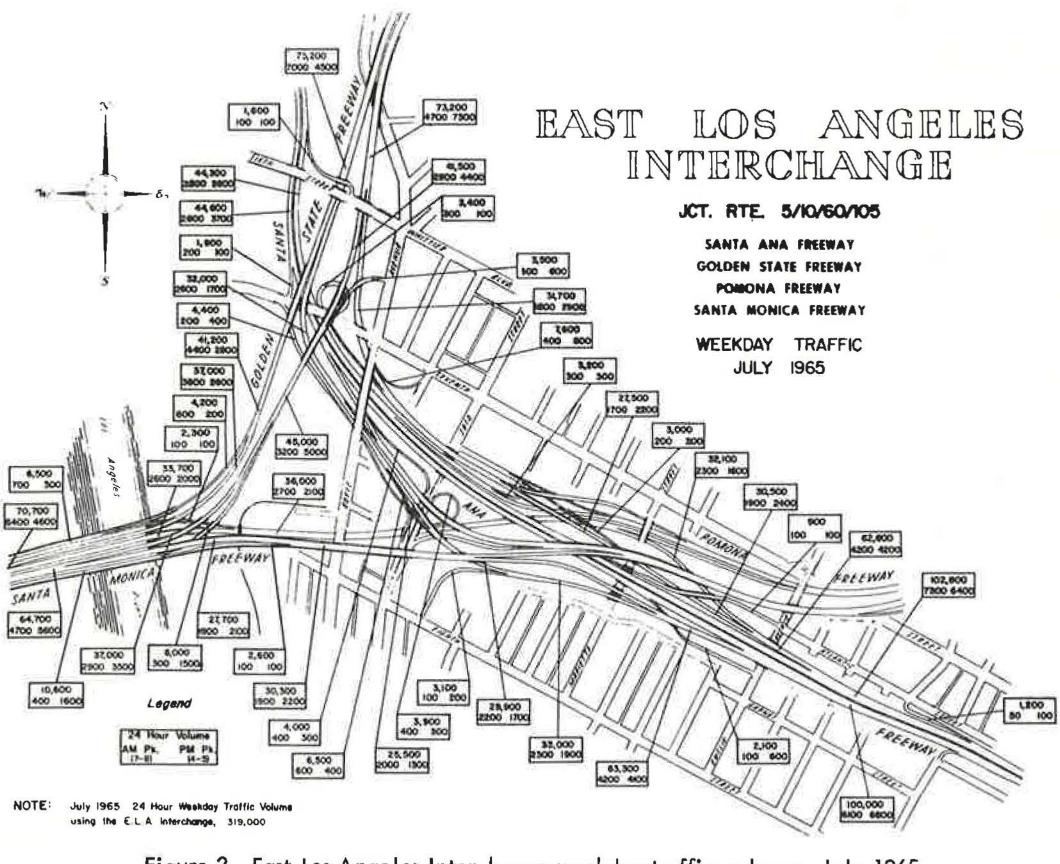


Figure 2. East Los Angeles Interchange weekday traffic volumes, July 1965.

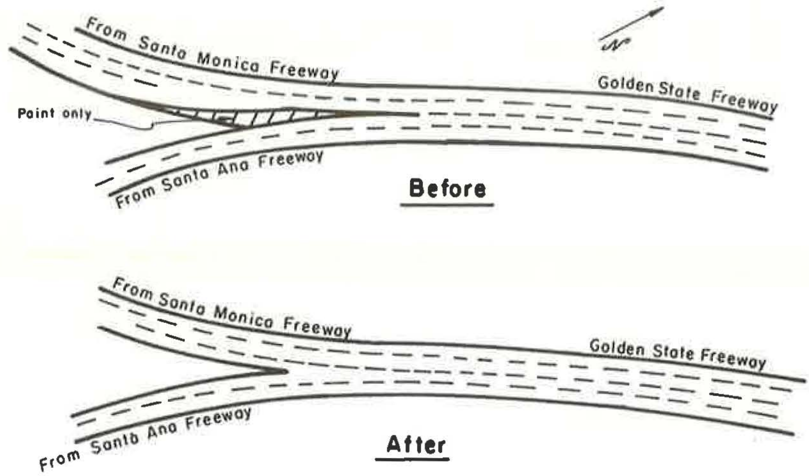


Figure 3. Striping of northbound merge.

This was because the sum of the traffic demand from the Santa Monica leg plus the ramp volume from the Santa Fe Avenue and Olympic Boulevard on-ramps (which included considerable mainline traffic bypassing the queue) exceeded the capacity of the 2 lanes that were available. Boyle Street off-ramp traffic (just upstream from the merge) was not enough to reduce demand to a 2-lane capacity.

Since there appeared to be some additional capacity on the Golden State Freeway, it was decided to stripe the Santa Monica approach for 3 lanes, which then became the conditions of the "after" study. "Before" and "after" striping are shown in Figure 3.

Results of this revision in striping are not as clear-cut as expected, but the figures indicate that capacity did increase and delay in the queue was less. One reason the improvement was not as large as it might have been is that during the "before" condition many vehicles used the painted-out lane despite the striping. This can be seen in Figure 4, which shows the "before" and "after" conditions at this location. Thus, there was not as great a change as would have occurred had drivers been physically restrained from using the third lane during the "before" period. Raised bars would have done this. One thing certain is that if raised bars had been placed in the painted-out lane, congestion would have been much worse, because the volume past this point during the "before" study (in excess of 4,400 vph) could not have been accommodated in two lanes.

Since demand and total flow between 4:00 and 6:00 p.m. were greater during the day of the "after" study, it is difficult to compare travel times directly. However, within a queue or back-up, higher capacity and better use of the available lanes were reflected by the lower density in the queue.

Table 1 gives volumes, travel (vehicle-miles), travel times (vehicle-minutes), and average speeds both upstream and downstream of the merging area. Vehicle-minutes are the product of the density readings (from aerial photographs), length of section, and the time interval between photographs. Actually, since density is number of vehicles per unit of length, the number of vehicles in each section at any point in time was multiplied directly by the time interval rather than first dividing the number by length of section and then multiplying back again.

From 3:00 to 4:00 p.m. (Table 1-a) the vehicle-minutes of travel were less in the "after" period, with most of the reduction on the Santa Monica approach where the capacity was increased by the striping changes. It should be noted that the demand was slightly less in the "after" period, as indicated by the decrease in vehicle-miles, but that the average travel time for all vehicles during the hour was reduced by 9 seconds. Average travel time was calculated by dividing vehicle-minutes of travel by the volume.

Distance between match line and Mateo St. is 2000 ft.
Number of vehicles in left 2 lanes is 74 or 195 per mile.
Average speed probably about 17 mph.

Number in right 2 lanes is 13 or 34 per mile which indicates free flow. With volume of 2000±, this calculates as 59 mph.

Before: Photos at approximately 4:25 p.m. Contact prints from 9 by 9 in. negatives, reduced for publication. On Santa Monica approach 3-lane pavement width available downstream of Boyle Ave. off-ramp. However, approach is narrowed to 2 lanes with paint only. Note that several vehicles are using this area anyway.

Number of vehicles between match line and Mateo St. is 55 or 145 per mile. Average speed probably about 24 mph.

Number in right 2 lanes is 17 or 45 per mile, which also indicates free flow. With volume of 2000±, this calculates as 44 mph.

After: Photos at approximately 4:25 p.m. Enlargements from 35-mm color transparencies. Santa Monica approach now striped for 3 lanes from Santa Fe on-ramp. Still lightly used but apparently increases capacity as travel time (vehicle-minutes) in back-up was less for equal or greater volume.

Figure 4. "Before" and "after" conditions at northbound merge of Santa Monica and Santa Ana Freeways.

TABLE 1

TRAVEL DATA—NORTHBOUND MERGE OF THE SANTA MONICA AND SANTA ANA FREEWAYS

Section No.	No. of Lanes	Length		Before					After				
		Feet	Miles	Volume	Vehicle-Miles	Average Density (veh/mile)	Vehicle-Minutes	Average Speed (mph)	Volume	Vehicle-Miles	Average Density (veh/mile)	Vehicle-Miles	Average Speed (mph)
(a) 3:00 to 4:00 p.m.													
Golden State Freeway (Downstream)													
26	4	600	0.114	5580	640	115	790	48	5370	610	137	940	39
27	4	650	0.123	5770	710	152	1120	38	5590	690	106	780	53
28	*	1120	0.212	5770	1220	148	1880	39	5590	1190	137	1740	41
Subtotals		2370	0.449		2570		3790	41		2490		3460	43
Santa Ana Approach													
29	2	490	0.093	2230	210	57	320	39	2160	200	54	300	40
30	2	270	0.051	2230	110	75	230	30	2160	110	33	100	66
Gap	2	380	0.072	2230	160	51	220	44	2160	160	42	180	52
31	2	820	0.155	1800	280	28	260	64	1750	270	49	460	35
Subtotals		1960	0.371		760		1030	44		740		1040	43
Santa Monica Approach													
32	**	710	0.134	3540	470	68	550	52	3430	460	60	480	58
34	3	800	0.152	3970	600	89	810	45	3780	570	84	770	45
35	3	400	0.076	3970	300	123	560	32	3780	290	81	370	46
37	2	670	0.127	3150	400	90	690	35	2950	370	71	540	42
38	2	930	0.176	3150	550	86	910	37	2950	520	61	650	48
Subtotals		3510	0.665		2320		3520	40		2210		2810	47
Santa Monica Freeway													
11	2	860	0.163	3150	510	76	740	42	2950	480	64	630	46
12	2	880	0.167	3150	530	59	590	53	2950	490	63	630	47
13 & 14	—	1640	0.311	3150	—	—	—	Free flow	2950	—	—	—	Free flow
Subtotals		3380	0.641		1040		1330	48		970		1260	46
Totals					6690		9670	41		6410		8570	45
(b) 4:00 to 5:00 p.m.													
Golden State Freeway													
26	4	600	0.114	6820	780	145	990	47	7000	800	156	1070	45
27	4	650	0.123	7070	870	164	1210	43	7350	900	157	1160	47
28	*	1120	0.212	7070	1500	207	2620	34	7350	1560	186	2370	39
Subtotals		2370	0.449		3150		4830	39		3260		4600	43
Santa Ana Approach													
29	2	490	0.093	2670	250	63	350	42	2920	270	77	430	38
30	2	270	0.051	2670	140	65	200	41	2920	150	42	130	69
Gap	2	380	0.072	2670	190	51	220	52	2920	210	46	200	63
31	2	820	0.155	2000	310	37	340	55	2300	360	49	460	47
Subtotals		1960	0.371		890		1110	48		990		1220	49
Santa Monica Approach													
32	**	710	0.134	4400	590	144	1160	31	4430	590	119	960	37
34	3	800	0.152	5010	760	180	1650	28	5040	770	137	1250	37
35	3	400	0.076	5010	380	206	940	24	5040	380	173	790	29
37	2	670	0.127	3440	440	151	1150	23	3590	460	159	1210	23
38	2	930	0.176	3440	610	170	1790	20	3590	630	138	1460	26
Subtotals		3510	0.665		2780		6690	25		2830		5870	29
Santa Monica Freeway													
11	2	860	0.163	3440	560	170	1660	20	3590	590	161	1570	22
12	2	880	0.167	3400	570	142	1420	24	3550	590	147	1470	24
13	2	860	0.163	3300	540	85	830	39	3450	560	135	1320	26
14	—	780	0.148	—	—	—	—	Free flow	—	—	—	—	Free flow
Subtotals		3380	0.641		1670		3910	26		1740		4360	24
Totals					8490		16540	31		8820		16050	33
(c) 5:00 to 5:30 p.m.													
Golden State Freeway													
26	4	600	0.114	6680	380	137	470	48	8100	460	158	540	52
27	4	650	0.123	6880	425	154	570	44	8370	515	184	680	45
28	*	1120	0.212	6880	730	214	1360	32	8370	885	248	1580	34
Subtotals		2370	0.449		1535		2400	38		1850		2800	40
Santa Ana Approach													
29	2	490	0.093	2700	125	68	190	39	3670	170	136	380	27
30	2	270	0.051	2700	70	91	140	30	3670	95	85	130	42
Gap	2	380	0.072	2700	95	69	150	39	3670	130	79	170	47
31	2	820	0.155	1900	145	45	210	43	2870	220	75	350	38
Subtotals		1960	0.371		435		690	38		615		1030	36
Santa Monica Approach													
32	**	710	0.134	4180	280	223	500	34	4700	315	147	590	32
34	3	800	0.152	4670	355	156	710	30	5150	390	167	760	31
35	3	400	0.076	4670	175	167	380	28	5150	195	228	520	23
37	2	670	0.127	3320	210	150	570	22	4040	255	136	520	29
38	2	930	0.176	3320	290	130	690	26	4040	355	133	700	30
Subtotals		3510	0.665		1310		2850	28		1510		3090	29
Santa Monica Freeway													
11	2	860	0.163	3320	270	163	800	20	4040	330	163	800	25
12	2	880	0.167	3320	275	214	1070	15	4040	335	196	980	21
13	2	860	0.163	3320	270	102	500	32	4040	330	174	850	23
14	4	780	0.148	—	—	—	—	Free flow	—	—	—	—	Free flow
Subtotals		3380	0.641		815		2370	21		995		2630	23
Totals					4095		8310	30		4970		9550	31

*"Before" = 4 lanes, "After" = 5 to 4 merge.
**"Before" = 2 lanes, "After" = 3 lanes.

Note: "Gap" indicates an area where it was impossible to photograph the freeway from the air because of the presence of overcrossings and the vehicle-minutes were calculated by averaging the densities in adjacent sections.

From 4:00 to 5:00 p.m. (Table 1-b) there was an overall decrease in travel time, although the demand volume increased. Travel time on the Santa Monica approach was reduced from 6,700 vehicle-minutes "before" to 5,900 vehicle-minutes "after" for a savings of 800 vehicle-minutes. The volume on the Santa Monica approach was 4,400 vehicles "before" and 4,430 "after." The total volume through the merge increased from 7,070 "before" to 7,350 "after." This savings of 800 vehicle-minutes is over a distance of 3,500 ft and is distributed to approximately 4,000 vehicles, which represents a savings of only about 12 seconds per vehicle. Average speed through the 3,500-ft section increased approximately 3 mph, which probably is not enough for the average driver to notice.

From 5:00 to 5:30 p.m. (Table 1-c) the vehicle-minutes of travel and the demand increased on all approaches. In the "after" period there is some doubt as to the validity of the traffic counts from 5:00 to 5:30, but it is known the volumes were higher than in the "before" period and that the average travel times did not change enough for the average driver to notice.

The net effect, then, of the striping changes at the northbound merge was to increase the capacity of the Santa Monica approach, thereby reducing the delay within the queue. This benefit was gained without a detrimental effect either to the Santa Ana approach to the merge or to the Golden State Freeway downstream of the merge.

Figures 5 through 8 show graphically the results of the study. Each graph shows density and volume on one of the approach legs. These density "contour" maps give a very good picture of operation and are an ideal way to study overall freeway performance. Densities less than 50 vehicles per mile per lane represent generally satisfactory flow conditions. Densities between 50 and 75 represent less than desirable flow, though high volumes can still be maintained at these densities. Densities greater than 75 indicate congestion and stop-and-go driving and usually mean there is some bottleneck downstream. Densities in or just downstream of a bottleneck will generally be from 45 to 60 vehicles per mile per lane. Average speed is obtained by simply dividing density into volume. If the volume is 2000 vehicles per hour and density is 50 vehicles per mile, then average speed is 40 mph.

Figures 5 and 6 show the "before" and "after" conditions on the Santa Monica approach. In Section 32 (the location of the striping changes) the densities shown are for the left two lanes only. These densities are considerably lower in the "after" period even though the demand was higher. This indicates that drivers are using the added lane in the "after" period.

Figures 7 and 8 show that conditions on the Santa Ana approach did not change appreciably except from 5:00 to 5:30 p.m., when the volumes during the "after" were greater than during the "before." It can be seen, though, that back-ups were very short. It can also be seen that there were no problems on the Golden State Freeway downstream of the merge. Densities seldom exceeded 50 vehicles per mile per lane during the "before" or "after" periods.

Southbound Merge

In the "before" condition (see Fig. 9) the Santa Monica Freeway approach to the southbound merge made the transition to 1 lane prior to its connection with the 2-lane Golden State Freeway approach leg.[1] The combined Santa Monica-Golden State approach (3 lanes) then merged with the 2-lane Santa Ana leg into 4 lanes. The "3-and-2-into-4" merge was accomplished by dropping the right-hand lane stripe, thus merging the right 2 lanes of the 3-lane Santa Monica-Golden State approach. This in effect gave the Santa Ana Freeway 2 free lanes.

During most of the day traffic demand on the 3 approach legs did not exceed capacity of the 4 downstream lanes. However, during much of the day traffic demand on the Santa Monica Approach did exceed the capacity of the single lane restrictions where it merged

[1]The original design had 2 lanes from the Santa Monica leg merge with 2 lanes from the Golden State leg into 3 lanes. However, this merge was short and because at one time traffic from the Santa Monica leg was light it was decided to narrow that approach to 1 lane by means of paint and raised bars.

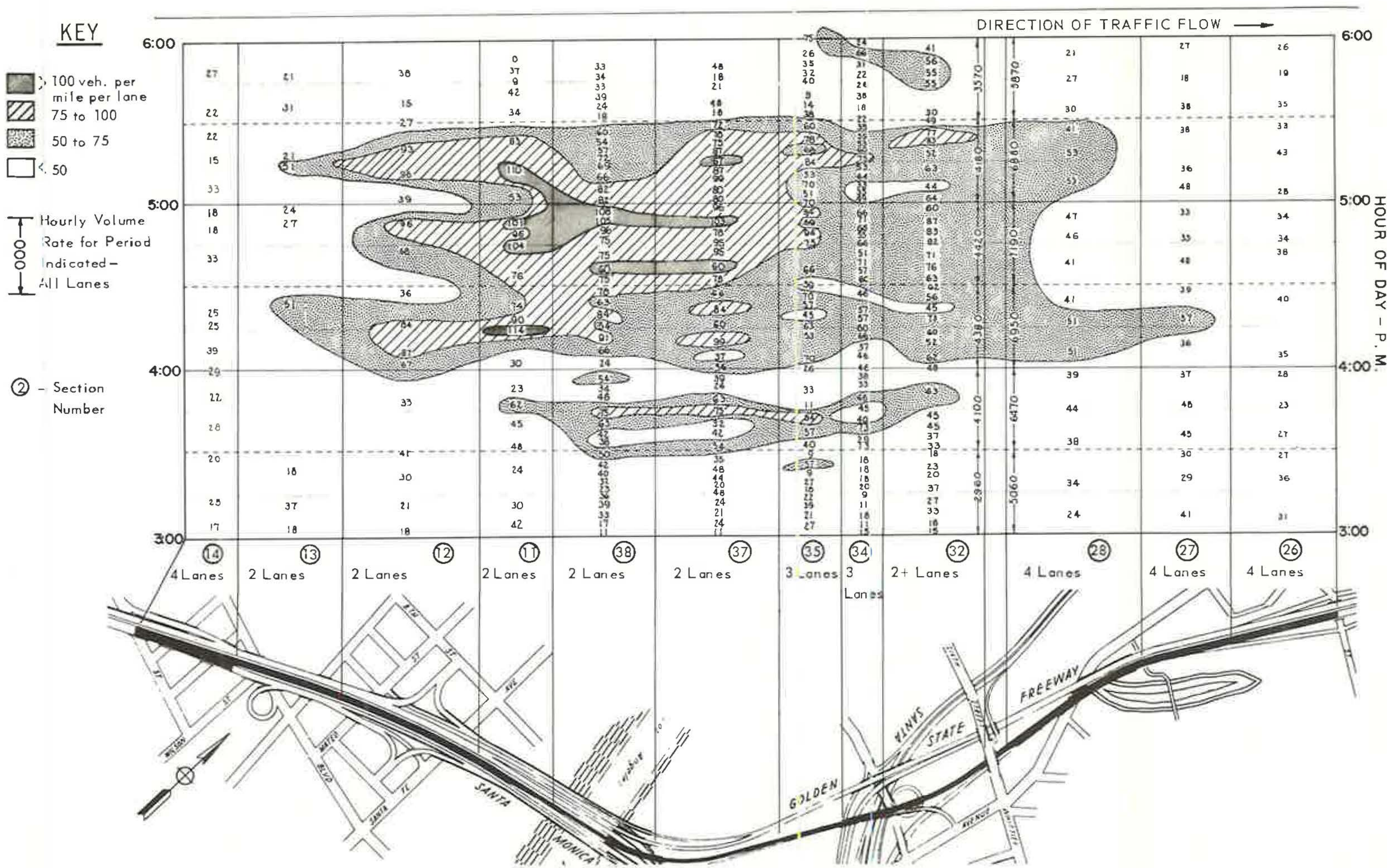


Figure 5. Northbound merge, "before"—densities and volumes on the Santa Monica and Golden State legs (Tuesday, May 18, 1965).

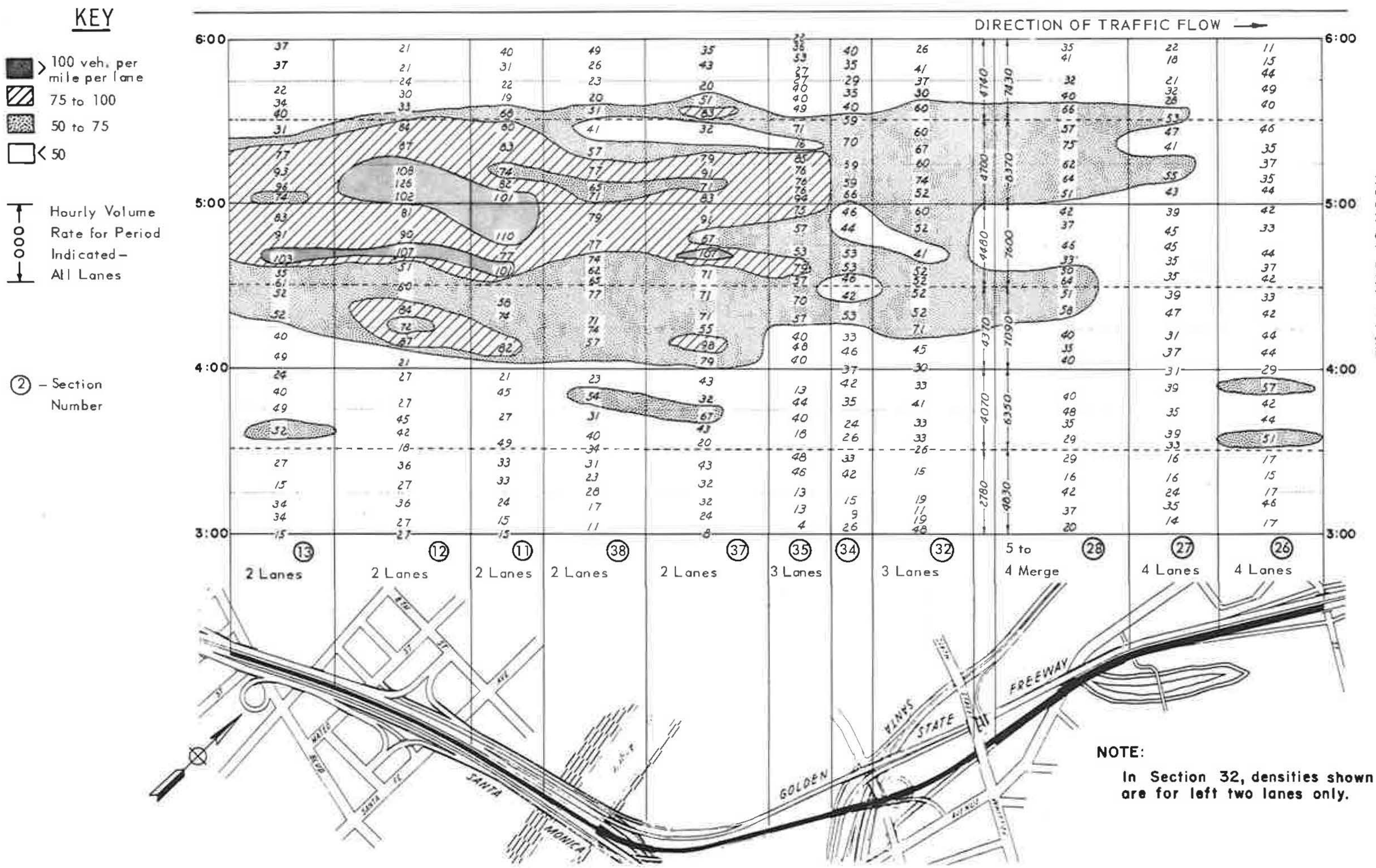


Figure 6. Northbound merge, "after"—densities and volumes on the Santa Monica and Golden State legs (Tuesday, Aug. 3, 1965).

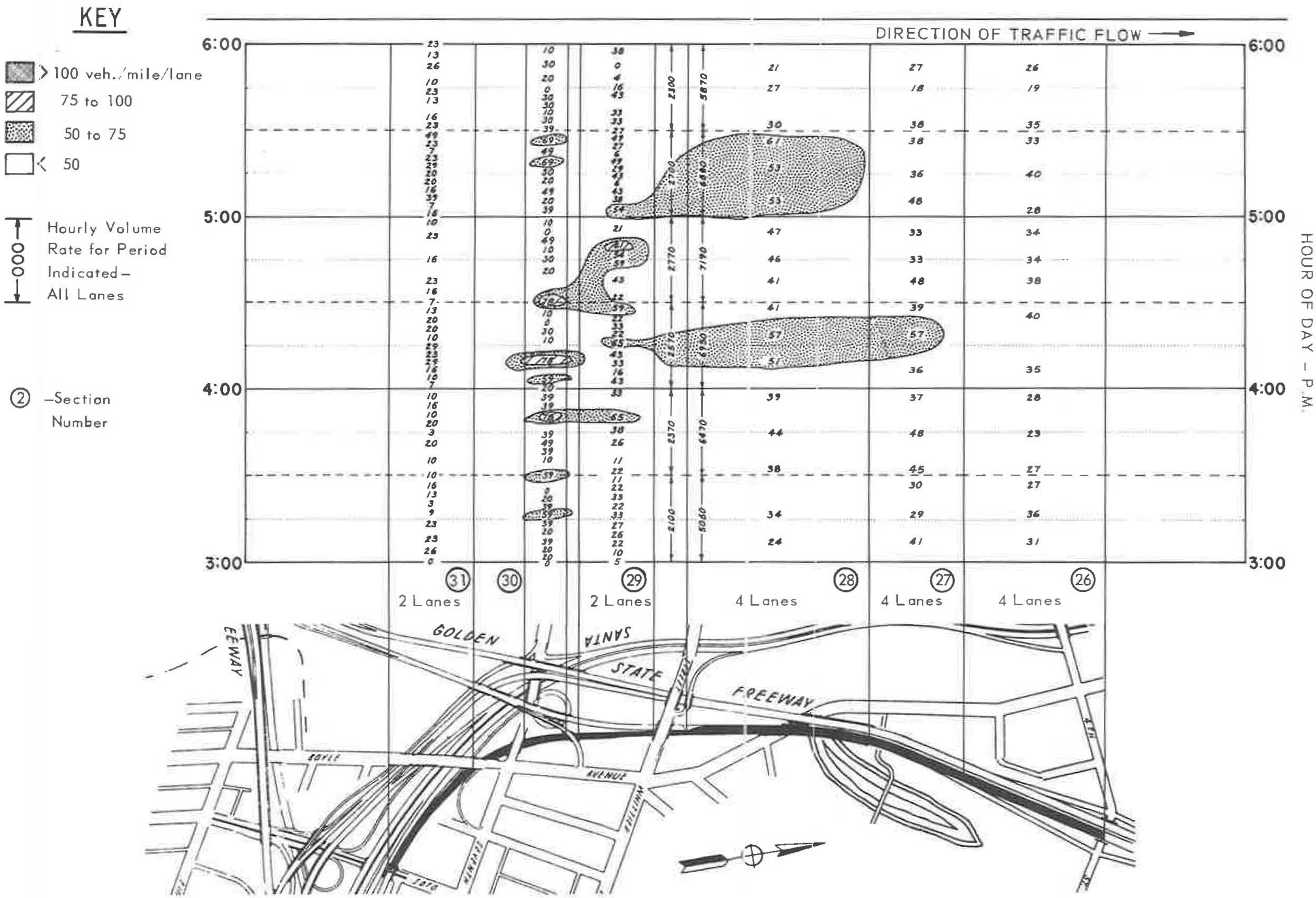


Figure 7. Northbound merge, "before"—densities and volumes on the Santa Ana and Golden State legs (Tuesday, May 18, 1965).

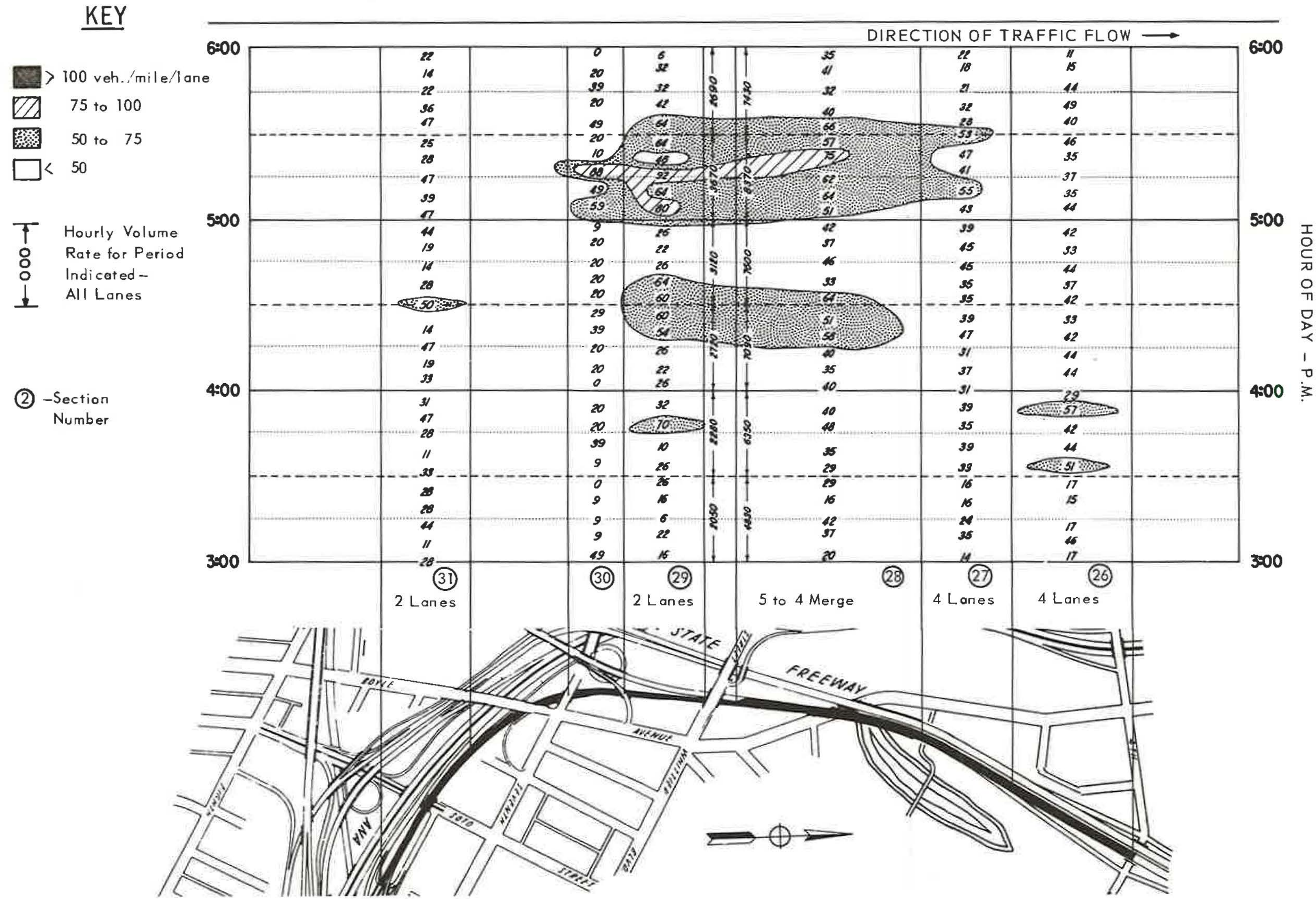


Figure 8. Northbound merge, "after"—densities and volumes on the Santa Ana and Golden State legs (Tuesday, Aug. 3, 1965).

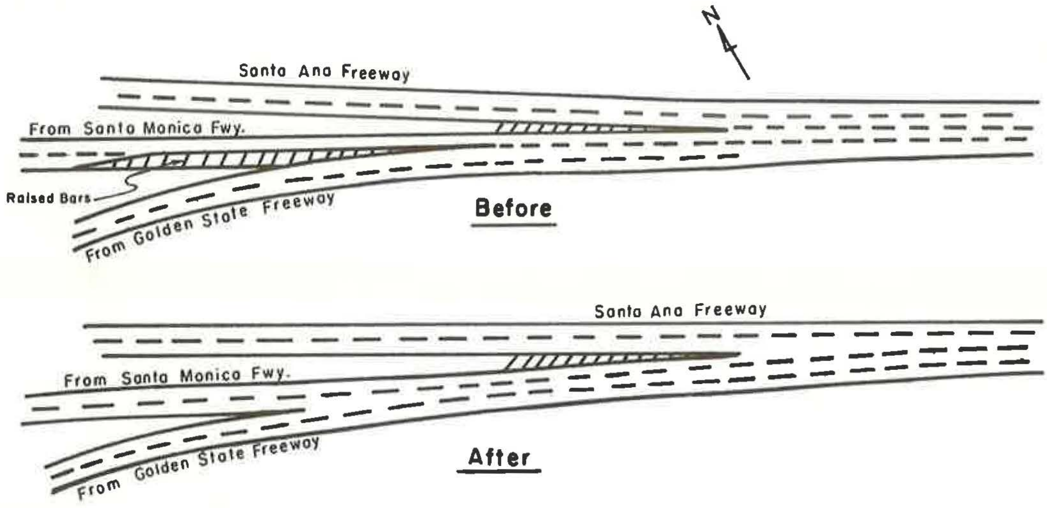


Figure 9. Striping of southbound merge.

with the ramp from the Golden State Freeway. Thus there would be back-ups and congestion during much of the day even though traffic on all other legs was free flowing. During peak periods total demand exceeded capacity of the Santa Ana Freeway further downstream and all approach legs would then back up. However, the back-up was much worse on the Santa Monica approach than on the other two upstream legs.

In order to eliminate some of this congestion and more equitably distribute delay among all approach legs, the raised bars were removed and the Santa Monica approach was striped for 2 lanes. The right lane of the Santa Monica approach then merged with the left lane of the 2-lane Golden State approach to form 3 lanes before joining the Santa Ana Freeway. The striping of the "3-and-2-into-4" merge was also altered so that the right lane of the Santa Ana merged with the left lane of the Santa Monica-Golden State approach. This constituted the "after" conditions. The "before" and "after" striping conditions are shown in Figure 9.

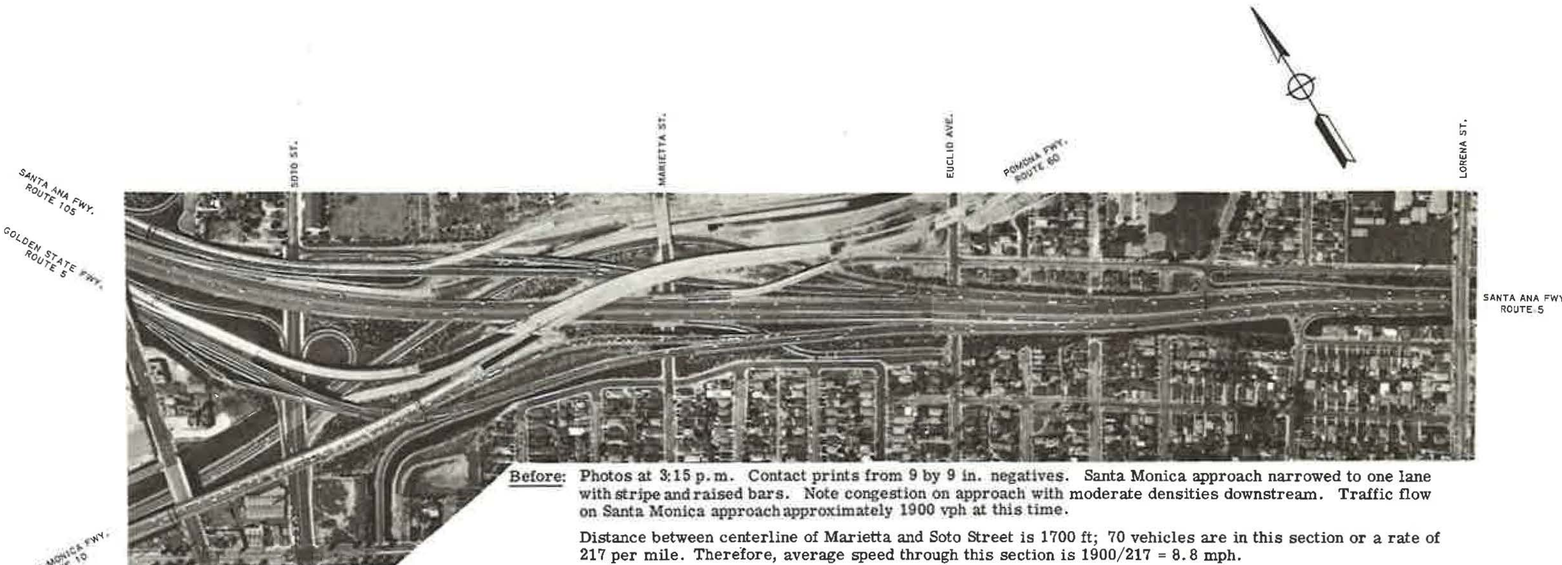
Operation both "before" and "after" was studied using ground counts and aerial photography. Figure 10 shows aerial photos with a thumbnail description of the study methods and results.

During off-peak hours all back-ups on the Santa Monica approach were eliminated and during peak hours when the 4-lane section downstream is the bottleneck, delay is more equitably distributed among all 3 approach legs in the "after" period.

Looking at the density contours in Figure 11, it can be seen that on the Santa Monica approach there was severe congestion (high densities) even at the start of the study (3:00 p.m. on a Tuesday, which is about the most lightly traveled weekday). It can also be seen that at this time there is unused space (densities are less than 50) on the downstream section, although the 4-lane section is very close to capacity.

Also at the end of the "before" study period (6:00 p.m.) there is congestion on the Santa Monica approach even though there is available room downstream.

Looking at the "after" condition, Figure 2, this congestion has been eliminated with equal or higher volumes. What this means is that a vehicle on the Santa Monica approach at 3:00 p.m. under the "before" conditions would be traveling at about 10 mph for about 3,000 ft. During the "after" period this vehicle would be traveling at about 45 mph. Thus, this vehicle would save over 3 minutes.



Before: Photos at 3:15 p.m. Contact prints from 9 by 9 in. negatives. Santa Monica approach narrowed to one lane with stripe and raised bars. Note congestion on approach with moderate densities downstream. Traffic flow on Santa Monica approach approximately 1900 vph at this time.

Distance between centerline of Marietta and Soto Street is 1700 ft; 70 vehicles are in this section or a rate of 217 per mile. Therefore, average speed through this section is 1900/217 = 8.8 mph.

After: Photos at 3:15 p.m. Enlargements from 35-mm color transparencies. Santa Monica approach given 2 lanes which merge with 2-lane Golden State approach into 3 lanes. Note absence of congestion. Traffic flow on Santa Monica approach approximately 2000 vph at this time.

There are 13 vehicles between Marietta and Soto Street or 40 veh/mile. Therefore, the average speed is 50 mph and each vehicle saves 1 min 48 sec traversing this 1700-ft section.

Figure 10. "Before" and "after" conditions at Southbound merge of Golden State, Santa Monica, and Santa Ana Freeways.

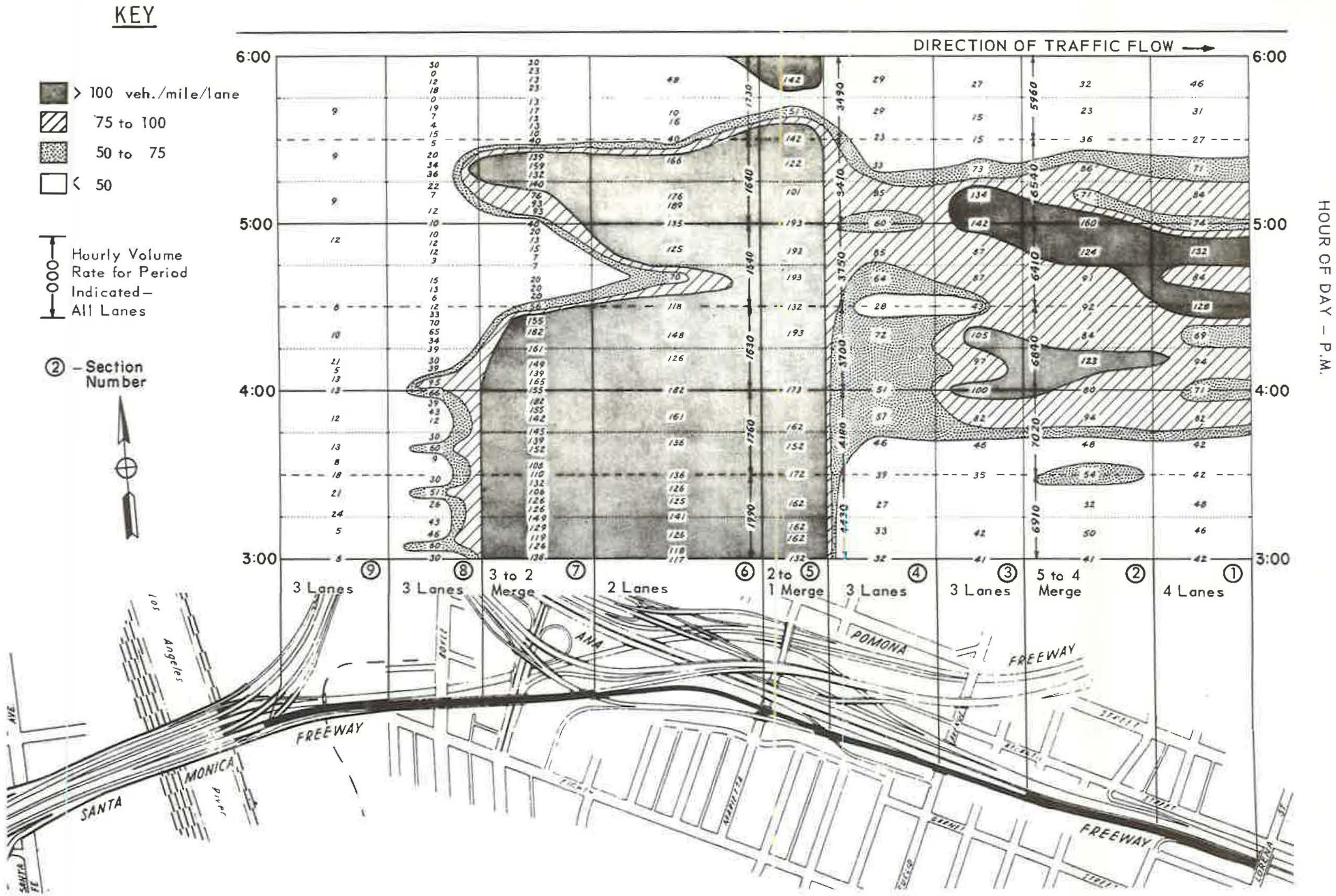


Figure 11. Southbound merge, "before"—densities and volumes on Santa Monica and Santa Ana legs (Tuesday, May 18, 1965).

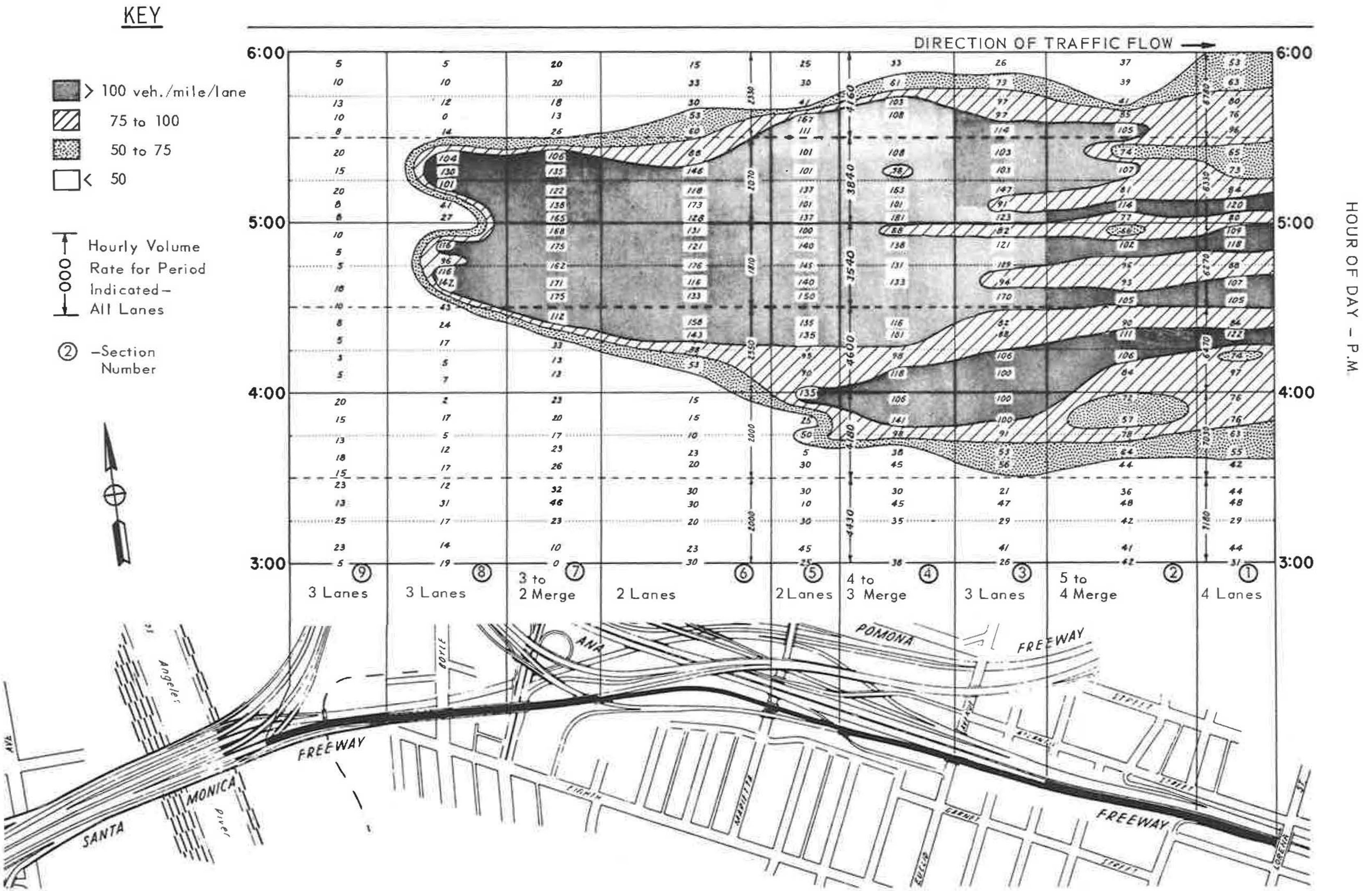


Figure 12. Southbound merge, "after"—densities and volumes on Santa Monica and Santa Ana legs (Tuesday, Aug. 3, 1965).

Table 2 gives travel times, volumes, and average speeds on the approaches to the southbound merging area and downstream of the merge.[2] From 3:00 to 4:00 p.m. (Table 2-a) there were 3,700 vehicle-minutes less in the "after" period with approximately the same total volume. Based on currently used monetary values for time (3 cents per minute) this represents slightly over $100 per day in user savings. While there was an average time savings for vehicles on the Santa Monica Freeway approach of slightly over 3 minutes per trip, there also was an increase of approximately 0.5 minute per trip for vehicles on the Santa Ana approach.

From 4:00 to 5:00 p.m. (Table 2-b) something happened downstream that caused the total travel time to be much greater during the "after" period. This was not caused by an increased demand during the peak, but primarily because of a reduction in capacity on the 4-lane section downstream where, due to some unknown conditions, the volume from 4:00 p.m. to 5:00 p.m. dropped from about 6,900 to 6,400 vehicles.[3]

However, the distribution of the delay was such that no one approach suffered appreciably at the expense of another (although in some cases this may not be a desirable objective). Excess travel times per vehicle during the hour from 4:00 to 5:00 p.m. were 3 minutes, 25 seconds; 3 minutes, 34 seconds; and 3 minutes, 40 seconds for vehicles on the Santa Ana, Golden State, and Santa Monica Freeway approaches respectively. (This is based on a "normal" travel speed of 45 mph and travel to the merge point of the Santa Ana and Santa Monica-Golden State.)

There is, however, one disadvantage resulting from the longer back-up on the Santa Ana approach. On the day of the "after" study, this back-up extended slightly north of 6th Street. Because of the longer back-up, traffic on the Santa Ana Freeway bound for the 7th and 8th Streets and Euclid Avenue off-ramps becomes involved to a certain extent in the delay, thus tending to increase overall delay. Back-ups on the Santa Monica and Golden State do not ordinarily reach a point where there are upstream off-ramps.

Capacity of the Santa Monica approach merge to one lane ("before" condition) was 2,000 vehicles per hour when there were no downstream restrictions. An all-day count (Sept. 15, 1965) showed hourly volumes as high as 2,250 vph. Had this approach still been reduced to 1 lane there would have been congestion at the merge for approximately 6 hours during the period 6:00 a.m. to 4:00 p.m. It is estimated that resultant delay would have been 37,000 vehicle-minutes, and there would have been approximately 9,000 vehicles delayed with an average delay of 4.1 minutes per vehicle (37,000/9,000). The maximum delay to any one vehicle would have been 9.7 minutes. At 3 cents per minute this delay represents over $1,000 per day in time value alone and indicates the large benefits that can be obtained from small increases in capacity, and conversely shows the large penalties that can be imposed by incorrect placement of pavement markings.

It was thought that when the Pomona Freeway was opened to traffic that it might carry sufficient volume so that the Santa Monica approach to the southbound merge could be

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[2] In Table 2 there are several places where the speed is indicated with a triple asterisk. When volume and density are low and speed is high, there are not very many vehicles in each photograph. For example, when the mean travel speed is around 60 mph and the rate of flow is 30 vehicles per minute in 2 lanes, a section 880 ft long will on the average contain only 5 vehicles. The actual observed number can vary from 1 or 2 to 10 vehicles, depending on chance, and considerable variability in calculated speed can result unless the calculated speed is based on a large number of observations (photographs). In Table 2, calculated speeds that are unrealistic because of this chance variability are marked with a triple asterisk. The way to avoid this would be to increase the number of observations by reducing the interval between photographs. In the present study, the interval was about 6 minutes, resulting in 10 observations per hour. This is plenty when congestion exists, and is sufficient even for free flow when calculating travel time for the entire system of roadways, but it is not sufficient for accurate determination of travel time for short sections. However, when free flow exists, it is not important to know speeds accurately and in fact we would not normally make this kind of a study where free flow exists and there is no problem.

[3] While the purpose of this paper is not to discuss the theory of delay, etc., it is interesting to note that a drop in capacity from 6900 and 6400 would in itself increase travel time by 15,000 vehicle-minutes in 1 hour, and this is roughly the difference shown in Table 2-b.

TABLE 2

TRAVEL DATA—SOUTHBOUND MERGE OF THE SANTA ANA, GOLDEN STATE, AND SANTA MONICA FREEWAYS

Section No.	No. of Lanes	Length		Before					After				
		Feet	Miles	Volume	Vehicle-Miles	Average Density (veh/mile)	Vehicle-Minutes	Average Speed (mph)	Volume	Vehicle-Miles	Average Density (veh/mile)	Vehicle-Minutes	Average Speed (mph)
(a) 3:00 to 4: 00 p.m.													
Santa Ana Freeway (Downstream)													
1	4	630	0.120	7150	860	214	1540	33	7100	850	206	1480	34
2	5 to 4	870	0.165	6970	1150	215	2130	32	6950	1150	213	2110	33
Subtotals	merge	1500	0.285		2010		3670	33		2000		35901	33
Santa Monica and Golden State Approach													
3	3	600	0.114	4250	480	149	1020	28	4420	500	168	1150	26
4	3	700	0.133	4250	570	102	810	42	4420	590	185	1480	24
Subtotals		1300	0.247		1050		1830	34		1090		2630	25
Santa Monica Approach													
5	*	520	0.099	1880	190	157	930	12	2000	200	74	440	27
6	2	1050	0.198	1880	370	292	3470	7	2000	400	42	500	48
7	3 to 2 merge	800	0.152	1880	290	265	2420	7	2000	300	50	460	40
8	3	730	0.139	1880	260	103	860	18	2000	280	44	370	45
9	3	700	0.133	1760	230	46	370	38	1820	240	53	420	34
10	3	725	0.138	1760	240	35	290	50	1820	250	37	310	48
Subtotals		4525	0.859		1580		8340	11		1670		2500	40
Golden State Approach													
17	2	520	0.099	2430	240	54	320	45	2420	240	88	520	28
18	2	920	0.174	2430	420	54	560	45	2420	420	50	520	49
Gap	2	680	0.129	1980	260	46	360	43	2050	260	44	340	47
19	2	520	0.099	1980	200	39	230	51	2050	200	39	230	53
Gap	2	400	0.076	2280	170	46	210	49	2450	190	46	210	53
41	2	550	0.104	2280	240	51	320	44	2450	250	53	330	46
40	2	770	0.146	2280	330	50	440	45	2450	360	41	360	59
Subtotals		4420	0.827		1860		2460	45		1920		2470	47
Santa Ana Approach													
22	2	600	0.114	2770	320	79	540	35	2830	320	110	750	26
23	2	1200	0.227	2770	630	61	830	45	2830	640	107	1460	26
Gap	3	570	0.108	2880	310	54	350	53	3010	330	83	540	36
24	3	960	0.181	2880	520	45	490	***	3010	540	58	630	52
25	3	780	0.148	2880	430	65	580	44	3010	450	75	670	40
25a	3	440	0.084	2880	240	64	320	45	3010	250	75	380	40
Gap	3	440	0.084	3270	270	64	320	51	3280	280	73	370	44
39	3	720	0.137	3370	460	64	530	52	3380	460	71	580	48
Subtotals		5710	1.083		3180		3960	48		3270		5380	36
Totals					9680		20260	29		9950		16570	36
(b) 4:00 to 5:00 p.m.													
Santa Ana Freeway (Downstream)													
1	4	630	0.120	6920	830	352	2730	18	6380	770	376	2810	16
2	5 to 4	870	0.165	6620	1090	408	4040	16	6100	1010	378	3750	16
Subtotals	merge	1500	0.285		1920		6770	17		1780		6560	16
Santa Monica and Golden State Approach													
3	3	600	0.114	3720	420	263	1800	14	4070	460	317	2170	13
4	3	700	0.133	3720	490	180	1440	21	4070	540	375	2590	13
Subtotals		1300	0.247		910		3240	17		1000		4760	13
Santa Monica Approach													
5	*	520	0.099	1580	160	168	1000	9	2180	220	266	1580	8
6	2	1050	0.198	1580	310	246	2920	6	2180	430	245	2910	9
7	3 to 2 merge	800	0.152	1580	240	172	1570	9	2180	330	225	2050	10
8	3	730	0.139	1580	220	89	740	18	2180	300	176	1470	12
9 & 10	3	1425	0.271	1480	400	24	390	61	2050	560	26	430	***
Subtotals		4525	0.859		1330		6620	12		1840		8440	13
Golden State Approach													
17	2	520	0.099	2140	210	42	250	51	1890	190	247	1470	8
18	2	920	0.174	2140	370	33	340	***	1890	330	221	2310	9
Gap	2	680	0.129	2010	260	31	240	***	1810	230	182	1410	10
19	2	520	0.099	1530	150	29	170	53	1400	140	143	850	10
Gap	2	400	0.076	1530	120	33	150	48	1400	110	97	440	19
41	2	550	0.104	1930	200	37	230	52	1800	190	48	300	38
40	2	770	0.146	1930	280	38	330	51	1800	260	27	240	***
Subtotals		4420	0.827		1590		1710	56		1450		7020	12
Santa Ana Approach													
22	2	600	0.114	3210	370	160	1090	20	2600	300	225	1540	12
23	2	1200	0.227	3210	730	97	1320	33	2600	590	210	2860	12
Gap	3	570	0.108	3470	370	79	510	44	3230	350	242	1570	13
24	3	960	0.181	3470	630	63	680	56	3230	580	277	3010	12
25	3	780	0.148	3470	510	64	570	54	3230	480	230	2040	14
25a	3	440	0.084	3470	290	78	390	44	3230	270	228	1150	14
Gap	3	440	0.084	3860	320	79	400	48	3760	320	224	1130	17
39	3	720	0.137	3960	540	83	680	48	3860	530	208	1710	18
Subtotals	3	5710	1.083		3760		5640	40		3420		15010	14
Totals					9510		23980	24		9490		41790	14

* "Before" = 2 to 1 merge, "after" = 2 lanes.

*** See footnote 2 on p. 42.

Note: "Gap" indicates an area where it was impossible to photograph the freeway from the air because of the presence of overcrossings and the vehicle-minutes were calculated by averaging the densities in adjacent sections.

changed back to one lane. However, a midday count (Jan. 14, 1966) revealed that the Pomona Freeway is carrying a relatively small volume and that the demand on the Santa Monica approach to the southbound merge still exceeds the capacity of 1 lane during much of the day. Demand for the Pomona Freeway, however, will increase as it is extended to the east.

CONCLUSIONS

Providing the greatest width possible on each of the approach legs of the East Los Angeles Interchange resulted in a marked improvement in overall operation.

In any interchange, it is very difficult to match the number of lanes on each leg precisely to the predicted traffic. This is because it is not only difficult to predict the amount of traffic and changing patterns but because of short-term variability in traffic flow. An average flow rate of 2,000 vph, for example, will have periods (very short) with much higher flow. A merge to 1 lane may have a capacity of, say, 2,000 vph and traffic demand may average 2,000 also. But in spite of this there can be a queue at the merge at this time, because short-term flow rates in excess of capacity will start the back-up and from then on it will not clear out until demand drops well below capacity.

This is essentially what was occurring on the eastbound Santa Monica approach to the southbound Santa Ana Freeway and to a certain extent on the approach to the northbound Golden State Freeway. Forcing the Santa Monica Freeway traffic to the southbound merge into 1 lane permitted a high standard merge, which made driving at low volumes somewhat more comfortable and possibly safer (safety aspects of this change are not analyzed in this report). However, this caused considerable congestion, and it is concluded that the best overall service is provided by eliminating much of this congestion at the price of a somewhat substandard merge.

It is possible to construct an improved merge, but any improvements involving substantial cost should be delayed until a significant portion of the Pomona Freeway (Route 60) is opened to traffic, because this may significantly reduce the demand on the southbound approach from the Santa Monica to Santa Ana Freeway.

It was noted that alterations to the southbound merge resulted in longer back-ups upstream on the Santa Ana Freeway. In the "after" condition the right lane of the Santa Ana approach merged with the left lane of the combined Santa Monica-Golden State approach. While this arrangement created a more equitable distribution of delay to traffic on all three approaches, the longer back-up on the Santa Ana approach does cause some delay to Santa Ana traffic destined for off-ramps upstream of the merge. It may have been better to give the Santa Ana approach a slightly better break and thus reduce peak-hour total delay at the expense of some additional delay to the Santa Monica-Golden State traffic. This could have been done by leaving the striping of the merge from the combined Santa Monica-Golden State into the Santa Ana the same as it was in the before condition, so that the right 2 lanes of the Santa Monica-Golden State approach merge to a single lane, giving the Santa Ana essentially 2 free lanes.

During the "before" condition on the northbound merge to the Golden State Freeway an attempt had been made to convert a 3-lane approach to a merge into 2 lanes by striping out the right-hand lane with a solid white stripe. During low-volume periods this was fairly effective and drivers usually followed the striping since they were not caused any delay by doing so. During peak periods, however, when the 2 lanes were congested, many drivers disregarded the striping and used the third lane. This definitely increased capacity, thus reducing delay, and did not seem particularly hazardous because of slow peak-hour speeds. This illustrates one advantage of using a painted stripe to discourage traffic from using a hazardous lane or section. That is, when traffic is fast and the section perhaps really is hazardous the striping is effective, but when traffic is slow and thus the section is not particularly hazardous, enough drivers will cross the stripe to provide the needed extra capacity.

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This project was accomplished in cooperation with the U.S. Bureau of Public Roads. The opinions, findings, and conclusions expressed are those of the Division of Highways and not necessarily those of the Bureau of Public Roads.

Appendix

ANALYSIS OF FREEWAY TRAFFIC FLOW BY AERIAL PHOTOGRAPHY

Aerial photography, supplemented by traffic counts on the ground, provided data for use in two studies of freeway operations in the Los Angeles area. This method has been found to be an ideal way to study freeway operations because it provides all the necessary data easily and at a reasonable cost.

The procedure involves taking a continuous series of aerial photos along the freeway under study. The flight pattern is repeated over and over so that a given segment of freeway is photographed many times during the study period. By counting the number of vehicles in each photograph and knowing the time interval between each flight pass, it is possible to calculate density and vehicle-minutes of travel. It is assumed that there are no fluctuations in traffic flow between each successive photograph of a given segment. By incorporating traffic volume data from ground counts, average speeds may also be calculated.

Photography

For the East Los Angeles Interchange (reported here) and the Harbor Freeway Ramp Closure studies, 35-mm photography was used. Previous studies of this type utilized 9 by 9-in. black-and-white negatives (1), but it was found that 35-mm color slides projected on a screen provided sufficient detail for counting vehicles at only a fraction of the film cost of the 9 by 9's. Both 35-mm and 50-mm focal length lenses were used. Where the 35-mm lens was used, the flight altitude was approximately 3,000 ft and for the 50-mm lens, 4,000 ft, to provide a scale of approximately 1 in. = 2,000 ft on the color transparencies.

Each flight pass consisted of a series of overlapping photographs, taken at about 2,000-ft intervals (measured longitudinally along the ground). Allowing for turn-around time, each section on the East Los Angeles Interchange Study was photographed on an average of once every 6 minutes, and on the Harbor Freeway Study, once every 10 minutes. In order to measure the entire delay due to a bottleneck, it is important that each pass of the aircraft cover enough length along the freeway to extend beyond the maximum length of queue expected during the study period.

Analysis

For convenience of analysis, the highway under study was divided into sections approximately 1,000 ft in length. Because each slide covered approximately 3,000 feet on the ground, the entire length of a section usually was photographed on a single slide.

The slides were projected on a screen and the number of vehicles in each section were counted for each pass of the aircraft. The vehicle count divided by the product of the section length and number of lanes gave the density in vehicles per lane-mile. The vehicle-minutes of travel were obtained by multiplying the average number of vehicles in each pair of photos by the time interval between photos in minutes. Average speeds for each time interval were calculated by dividing the volumes (veh/hr) obtained from ground counts by densities (veh/mi) from analysis of the aerial photographs. [The average speed could also have been calculated by dividing the vehicle-miles (volume ×

TABLE A

HARBOR FREEWAY "AFTER" STUDY
SECTION 16 (length = 1740 feet), 3:00 to 4:00 p.m.

Flight No.	Time	Vehicle Count From Photographs	Average No. of Vehicles	Time Interval for Analysis (min)	Section Length (lane-mi)	Density (veh/lane-mi)	Travel (veh-min)
1	3:00	48				36	
			49.5	8	1.32		396
2	3:08	51				39	
			44.0	6			264
3	3:14	37				28	
			39.5	5			198
4	3:19	42				32	
			48.0	4			192
5	3:23	54				41	
			48.5	5			243
6	3:28	43				33	
			43.0	3			129
7	3:31	43				33	
			46.5	6			279
8	3:37	50				38	
			52.0	4			208
9	3:41	54				41	
			44.5	5			223
10	3:46	35				27	
			42.5	5			213
11	3:51	50				38	
			50.0	9			450
12	4:00	50					
				Total 60		Avg. 35	Total 2795

Volume (from counts) = 6920 veh/hour
Avg. density (4 lanes) = 4 x 35 = 140 veh/mi

$$\text{Avg. speed} = \frac{\text{Volume}}{\text{Avg. Density}} = \frac{6920}{140} = 49.4 \text{ mph}$$

length of section) by the vehicle-minutes of travel obtained from the analysis of photographs.]

Table A gives the calculations involved in obtaining densities and vehicle-miles for a 1-hour period on one section of the Harbor Freeway Study. This is illustrative of the type of analysis used for each freeway section for both the East Los Angeles Interchange and Harbor Freeway studies.

Reference

1. Smith, Walter P., Jr. Use of Aerial Photography to Study Traffic Delay Due to a Lane Closure. California Division of Highways.

Determination of Merging Capacity and Its Applications to Freeway Design and Control

DONALD R. DREW, JOHANN H. BUHR, and ROBERT H. WHITSON,
Texas Transportation Institute, Texas A & M University

This paper presents a new approach to the determination of merging capacity and merging service volumes. For the first time in the literature, a method is presented of quantitatively determining the influence of entrance ramp geometrics on the capacity and level of service of a ramp-freeway merging area. The procedure enables the design engineer to evaluate rationally alternate designs and, if compromise is needed, to select the element or location where such compromise will be the least objectionable.

The capacity of a merging area is based on the interaction between the gap acceptance behavior of entrance ramp drivers and the availability of gaps on the freeway shoulder lane, while the service volumes suggested are developed from considerations of the ramp junction as a queuing system. This permits the provision of a level of service such that a ramp vehicle has a certain probability of finding the merging area empty. Another measure of level of service—the delay suffered by ramp vehicles—is treated and charts are presented for its determination.

The merging parameters developed are closely linked to the critical gap of the ramp-freeway junction. Through the study of merging operations at 29 entrance ramps across the continental United States, relationships are developed between the gap acceptance characteristic and certain design characteristics of an entrance terminal. This allows for estimation of the critical gap and hence for the capacity and service volume of an on-ramp, based on the length of acceleration lane, the angle of convergence and the shape of the acceleration lane. Appropriate charts are presented for estimation of the merging parameters and the influence of design characteristics. The application of the developed parameters to freeway control via the "gap acceptance" mode of control is also discussed.

•IN freeway design, the engineer is often faced with the problem of determining the capacity of a merging area. Existing procedures permit the estimation of outside freeway lane volumes, which are then subtracted from fixed control values to give allowable ramp service volumes (1, 2, 3, 4). These procedures take various traffic characteristics and ramp configurations into consideration, but do not account for the effects of the geometrics of the ramp terminal itself. As a result, it is usually assumed that the entrance facility does not suffer from design deficiencies such as a short acceleration lane, a high angle of entry, inadequate sight distance, or poor delineation. In practice, however, physical limitations will often preclude the development of ideal geometrics so that the designer is forced to use a substandard design. In this paper,

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the effect on the merging operation by two of these geometric variables, acceleration lane length and angle of entry, is analyzed and their application to the design and control of entrance terminals is discussed.

Parts of this paper appear in a series of papers (5, 6, 7, 8) on freeway merging that resulted from research undertaken by the Texas Transportation Institute and sponsored by the U.S. Bureau of Public Roads.

BACKGROUND

There are several well-known variations of a merging problem that arise where traffic in a secondary stream joins a primary flow stream. As the problem is sometimes formulated, one of the vehicles in the secondary stream merges with the primary stream whenever the headway to the next vehicle in the primary stream is greater than or equal to a constant value, T, or in the case of multiple entries, T + iT for i vehicles to merge with the primary stream. To describe this constant value, T, it is usually assumed that there exists a certain fixed gap size or one that the ramp driver will probably enter. The smallest gap that ramp drivers will accept with a certain probability has been termed the "critical gap." Several "critical" values have been discussed in the literature of Greenshields and Raff. Greenshields (9) defines the "acceptable average-minimum time gap" as a gap accepted by half the drivers. Raff (10) uses a slightly different parameter, the "critical lag." This parameter, as in the case of Greenshields' work, is a median value. If one assumes, and rightly so, that there exists a critical gap function, P(T), which describes the relative frequency of different size critical gaps among drivers, then there is associated with this function a median critical gap, referred to in this paper as T.

The principal use of gap acceptance parameters is to simplify the computation of the delay duration by permitting the assumption that all intervals shorter than the critical value (lag or gap) are rejected while all intervals longer are accepted. It has been suggested (11, 12) that the mean of the critical gap distribution, not the median, should be used in delay computation. Nevertheless, the "median critical gap" remains a practical parameter and will be referred to hereafter as the critical gap.

DETERMINATION OF MERGING PARAMETERS

Merging Capacity

The efficiency of traffic movement on the through lanes of an urban freeway is directly affected by the adequacy of the associated ramps. The proper design and placement of ramps on high-volume freeways is therefore imperative if those facilities are to afford fast, efficient, and safe operation. The development of such suitable designs depends to a large extent on the accurate determination of the capacity at the ramp junction, heretofore referred to as the merging capacity.

In the merging situation, the maximum number of ramp vehicles that can be accommodated in the shoulder lane is equivalent to the number of ramp vehicles that will use each available gap, assuming a continuous backlog of vehicles on the ramp (13). The concept of gap acceptance is therefore of major importance in considerations of the capacity of a merging area.

Consider a single, inexhaustible queue waiting to enter a random shoulder-lane traffic stream. If the passing time headway, t, is less than the critical gap, T, no ramp vehicle enters; if t is between T and T + T′ one vehicle enters; if t is between T + T′ and T + 2T′ two vehicles enter, etc. The ability of the outside freeway lane to absorb ramp vehicles per unit of time becomes

$$q_r = q \sum_{i=0}^{\infty} (i + 1)\, P[T + iT' < t < T + (i + 1)T'] \tag{1}$$

where q is the shoulder-lane flow.

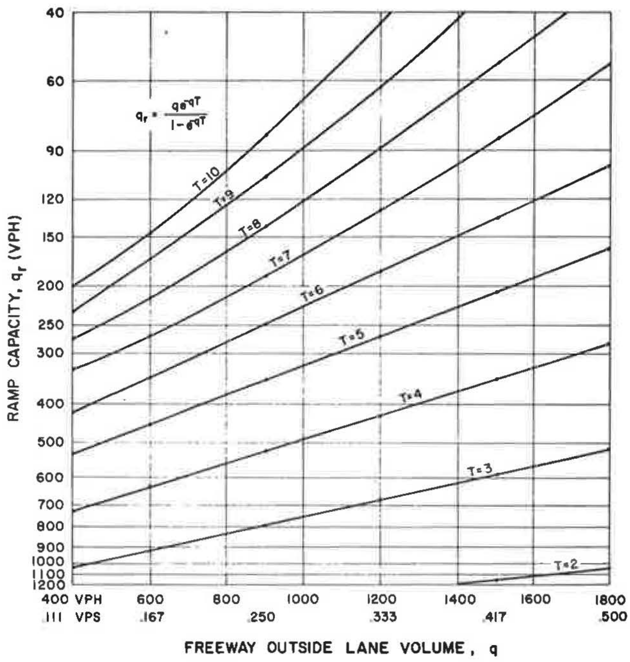

Figure 1. Possible capacity of merging areas.

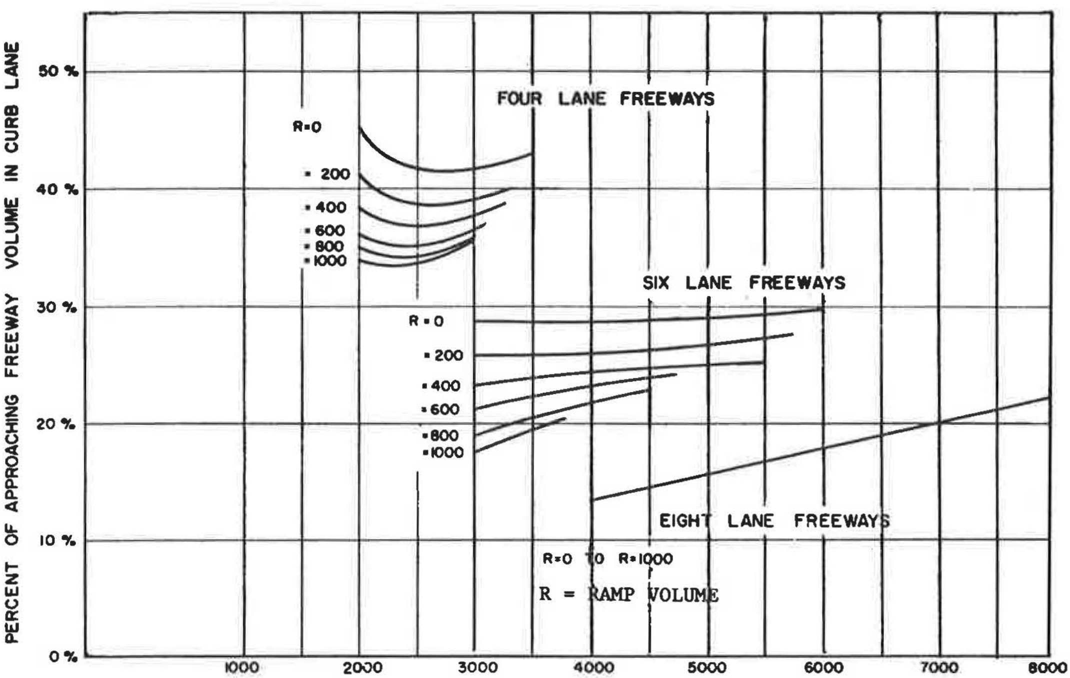

Figure 2. Relationship between percent of total freeway volume in the outside lane and freeway and ramp volumes.

If the distribution of gaps in the shoulder lanes, $f(t)$, is given by the negative exponential distribution

$$f(t) = qe^{-qt}$$

then it follows from Eq. 1 that

$$\begin{aligned}
q_r &= q\left[e^{-qT} - e^{-q(T + T')}\right] + 2q\left[e^{-q(T + T')} - e^{-q(T + 2T')}\right] + \ldots \\
&= qe^{-qT} + qe^{-q(T + T')} + qe^{-q(T + 2T')} + \ldots \\
&= qe^{-qT}\left(1 + e^{-qT'} + e^{-2qT'} + \ldots\right) \\
&= \frac{qe^{-qT}}{1 - e^{-qT'}} \qquad (2)
\end{aligned}$$

In a previous report (6) dealing with multiple entries it was concluded that double entries are more "sensitive" than single entries, and triple entries more "sensitive" than double entries to differences in gap sizes. Here "sensitivity" means the data show that the percent acceptance curves are steeper for triple entries than for double entries and steeper for double than for single entries. The percent acceptance curves show that at the 50 percentile acceptance level (the critical gap), T and T′ are approximately equal. Therefore, the expression for ramp capacity in Eq. 2 may be simplified, becoming

$$q_r = \frac{qe^{-qT}}{1 - e^{-qT}} \qquad (3)$$

Equation 3 is illustrated in Figure 1. In order to use the graph, it is necessary that the shoulder-lane volume, q, and the critical gap, T, be known. If an existing design is being evaluated, q, q_r and T can be measured. In the case of a proposed design, methods of estimating the percent of the total freeway volume in the shoulder lane are well documented (1). Variables that have been found to significantly affect q are the total freeway volume, the entrance ramp volume q_r, upstream ramp volume, downstream exit ramp volume, and distance to downstream exit ramp. Figure 2 is illustrative of the relationship between q and two of these variables—total freeway volume and entrance ramp volume, q_r (14).

Merging Service Volumes

As defined previously, the merging capacity is the maximum number of vehicles that can be accommodated with a continual backlog or queue of ramp vehicles. Whenever the opportunity occurs for r vehicles to enter the shoulder-lane stream, there must, of course, be at least r vehicles queued on the ramp to utilize this capacity potential. Although the delay or queue lengths associated with such a traffic condition could be excessive, they were not considered in the capacity analysis. However, in order to provide a certain level of service, the determination of merging service volumes must take delays or queue lengths into account.

In an earlier report in this series (6) expressions were derived for the mean and variance of the delay suffered by ramp vehicles in position to merge. They are

$$E(t)_a = \frac{e^{aqT} - \sum_{i=0}^{a} \frac{(aqT)^i}{i!}}{q \sum_{i=0}^{a-1} \frac{(aqT)^i}{i!}} \tag{4}$$

and

$$\sigma^2(t)_a = \frac{(a + 1)\left[e^{aqT} - \sum_{i=0}^{a+1} \frac{(aqT)^i}{i!}\right]}{aq^2T \sum_{i=0}^{a-1} \frac{(aqT)^i}{i!}} + E^2(t)_a \tag{5}$$

where a is the parameter of the Erlang distribution denoting the distribution of headways in the shoulder-lane stream.

Considering the ramp junction as a queuing system, the entrance ramp vehicles arrive at the junction at a rate, q_r, and are obliged to yield to the freeway traffic, thus forming a single line waiting for successive vehicles at the head of the queue to merge. If the moments of the distribution of time, f(t), spent by vehicles at the head of the queue are given by Eqs. 4 and 5, the expressions for some useful queuing parameters may be developed.

Let n_0, n_1 denote the ramp queue lengths immediately after two successive ramp vehicles C_0, C_1 have merged. Let t be the service time of C_1 and r be the number of ramp vehicles arriving while C_1 is being served. If a random variable δ is introduced such that $\delta = 1$ if $n_0 = 0$ and $\delta = 0$ if $n_0 \neq 0$, then it follows that

$$n_1 = n_0 + r - 1 + \delta \tag{6}$$

It is to be noted from the definition of δ that $\delta^2 = \delta$ and that $n_0\delta = 0$, and hence, from Eq. 6, on taking expected values, we obtain

$$E(n_1) = E(n_0) + E(r) - 1 + E(\delta) \tag{7}$$

If the system is assumed to be in a state of statistical equilibrium, then

$$E(n_1) = E(n_0) = E(n)$$

and

$$E(r) = q_r E(t)_a$$

Thus, substituting in Eq. 7,

$$E(\delta) = 1 - q_r E(t) \tag{8}$$

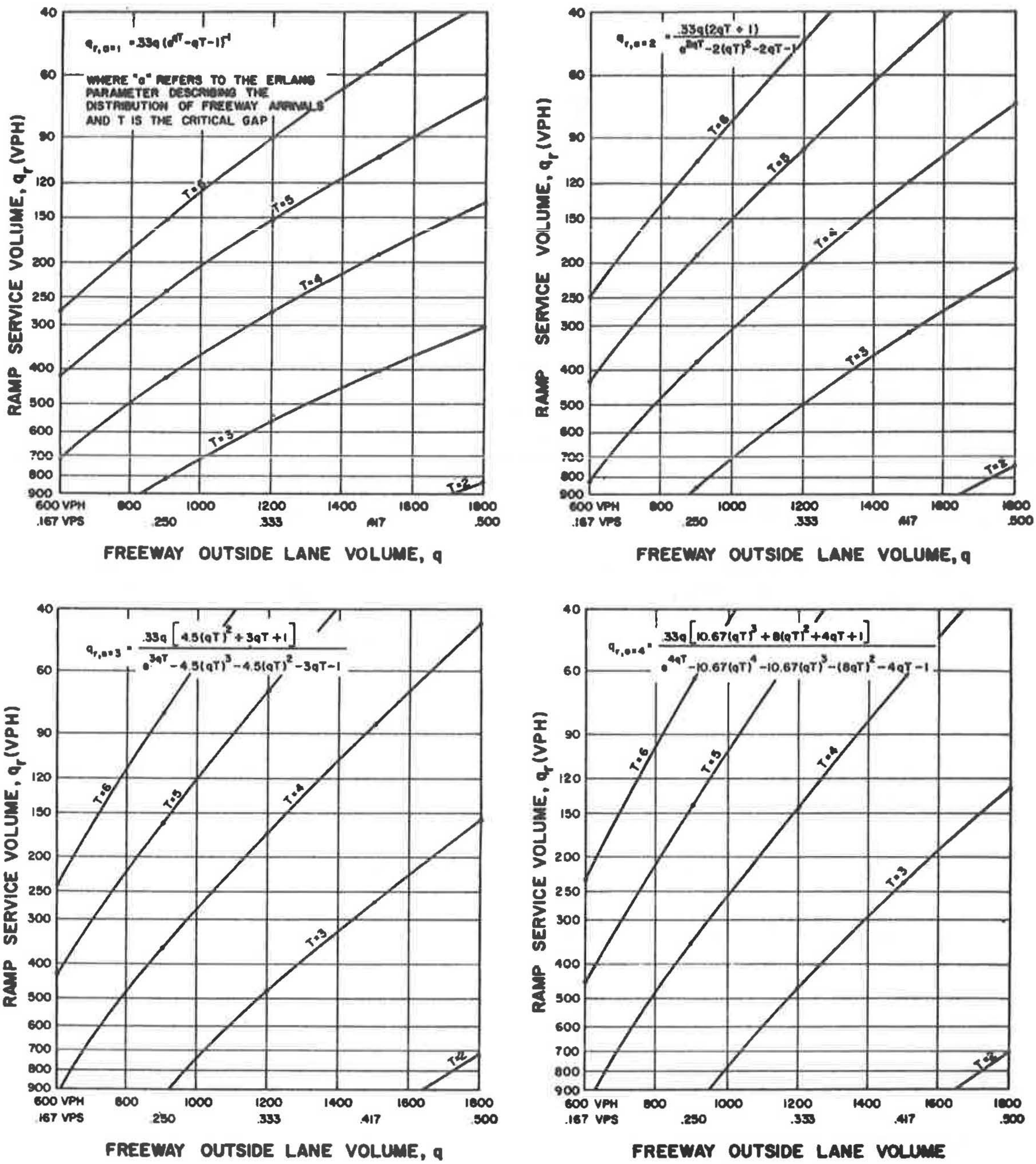

Figure 3. Maximum service volume q_r for a ramp arrival to have a probability of 0.67 of finding no ramp vehicles in the merging area.

However, we also know that by definition

$$E(\delta) = \sum_{\delta = 0}^{1} \delta P(\delta) = P(\delta = 1) = P(n_0 = 0) = P_0 \qquad (9)$$

Equating Eq. 8 to Eq. 9, one finds that

$$q_r = \frac{(1 - P_0)}{E(t)_a} \qquad (10)$$

The ramp flow, q_r, may be interpreted as a ramp service volume as opposed to ramp capacity; P_0 is the probability of a ramp arrival finding the merging area empty, and as such affords a measure of level of service. Substituting Eq. 4 in Eq. 10 and arbitrarily allowing P_0 to equal 0.67 provides the basis for the ramp service volume curves in Figure 3. For example, for a = 1, the ramp service volume is given by

$$q_r = 0.33q\left(e^{qT} - qT - 1\right)^{-1} \qquad (11)$$

Figure 3 illustrates the relationship between the freeway shoulder-lane volume, q, the headway distribution of the shoulder-lane stream as defined by the Erlang parameter, a, the critical gap, T, and the ramp volume, q_r. Consider an entrance ramp operating with a critical gap of 4.0 seconds and with the distribution of freeway traffic conforming to an Erlang distribution with a = 2. It is apparent that the sum of the coordinates of any point on the line T = 4 in the graph a = 2 of Figure 3 describes the merging service volume for that ramp. For example, the point described by q = 1500 and q_r = 120 tells us that the merging service volume is 1620 and that under these operating characteristics, a ramp arrival has a 67 percent chance of finding the ramp empty or a 33 percent chance of finding a vehicle ahead of it trying to merge.

In the design of a new facility, however, the engineer is confronted with a set of assigned volumes and has no knowledge of the value of T and a, and is therefore at a loss as to which curve or even which set of curves to use for determining the service volume of a proposed design. Toward this end, the collected data were analyzed with the objective of formulating relationships that would be useful to the designer in that they should allow the prediction of the Erlang parameter, a, and the initial gap, T, from information that would normally be available.

The Erlang parameter is largely affected by the volume level. To be sure, there are several other variables, such as alignment, grade, and other environmental elements, that affect the value of a. However, in the absence of any knowledge of these variables, the curve shown in Figure 4 has been found to approximate the value of a as related to the freeway volume. This curve can be used in con-

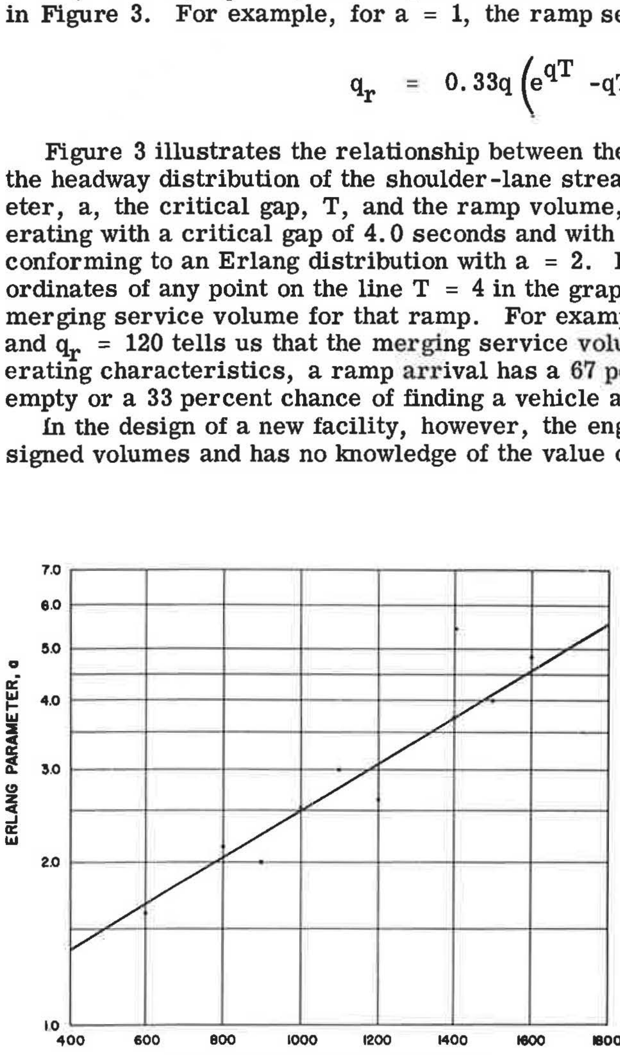

Figure 4. Approximate value of Erlang a as related to the freeway outside lane volume.

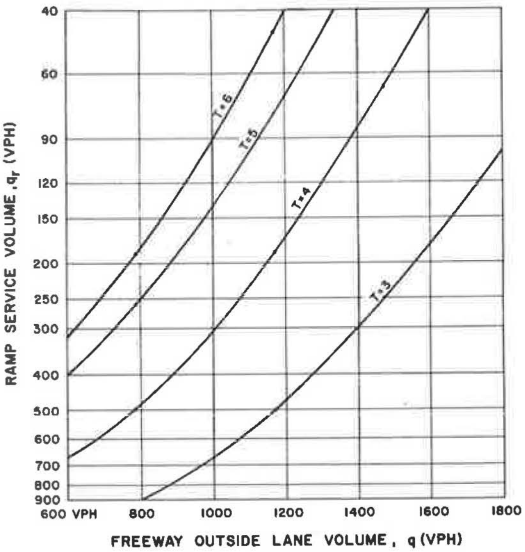

Figure 5. Maximum service volume q_r for a ramp arrival to have a probability of 0.67 of finding no ramp vehicles in the merging area.

junction with the relationships shown earlier in Figure 3 to develop the set of curves shown in Figure 5, which relates the ramp service volume to the outside freeway lane volume and the critical gap, T, eliminating the need for a knowledge of the Erlang parameter, a, of the freeway traffic time headway distribution. The critical gap, T, can be estimated from the geometrics of the entrance terminal. Before this is treated in detail, some other measures of level of service will be discussed.

The mean queue length confronting an arriving ramp vehicle and its delay (time in the system) are additional measures of the level of service afforded ramp traffic. Expressions for these parameters are obtainable using the techniques in Eqs. 6 through 11. Squaring both sides of Eq. 6 and taking expected values as before leads to

$$E(r - 1)^2 + E(\delta^2) + 2E[n_0(r - 1)] + 2E[\delta(r - 1)] = 0$$

which reduces to

$$E(n_0) = \rho + \frac{E(r^2) - \rho}{2(1 - \rho)} \qquad (12)$$

where $\rho = q_r E(t)$.

It is now necessary to calculate $E(r^2)$, the second moment of the number of arrivals in the service time, t, making use of its relationship to the mean and variance in arrivals. Assuming that ramp arrivals are Poisson and remembering that "averaging" here must be carried out with respect to both r and the service time, t, we have

$$E(r^2) = q_r E(t) + q_r^2 E(t^2)$$

and, considering the relationship between the first two moments and the variance

$$E(t^2) = \sigma^2(t) + E^2(t)$$

then

$$E(r^2) = \rho + \rho^2 + q_r^2 \sigma^2(t) \qquad (13)$$

Substituting Eq. 13 in Eq. 12 gives the expected queue length on the ramps as

$$E(n) = \rho + \frac{q_r^2 \sigma^2(t) + \rho^2}{2(1 - \rho)} \qquad (14)$$

If w is the waiting time (before merging) of C_1, then n_1 ramp vehicles arrive in time $t + w$. Thus, since the mean arrival rate is q_r,

$$E(n) = q_r E(t + w)$$

and

$$E(w) = [E(n)/q_r] - E(t) \tag{15}$$

It follows that the mean wait in the system for a ramp vehicle is

$$E(v) = E(n)/q_r \tag{16}$$

The curves for Eq. 16 are plotted in Figure 6 in terms of the shoulder-lane volume, q, and the ramp volume, q_r. The distribution of headways on the shoulder lane is assumed to be random (a = 1) with critical gap values of T = 3, 4, 5, and 6 seconds.

Estimation of the Critical Gap

As indicated previously, the critical gap, T, must be estimated in order to find the capacity and service volume of a proposed design. This estimation must, of course, be based on the geometrics of the merging area.

The elements of good design for an entrance ramp junction are well documented. They are, to name a few: (a) adequate length for drivers to accomplish merging; (b) a flat angle of approach that aligns the driver along an easy and natural path into the acceleration lane; (c) good visibility to allow the entrance ramp driver to judge and accept a freeway gap with a minimum of indecision; and (d) a clearly marked and

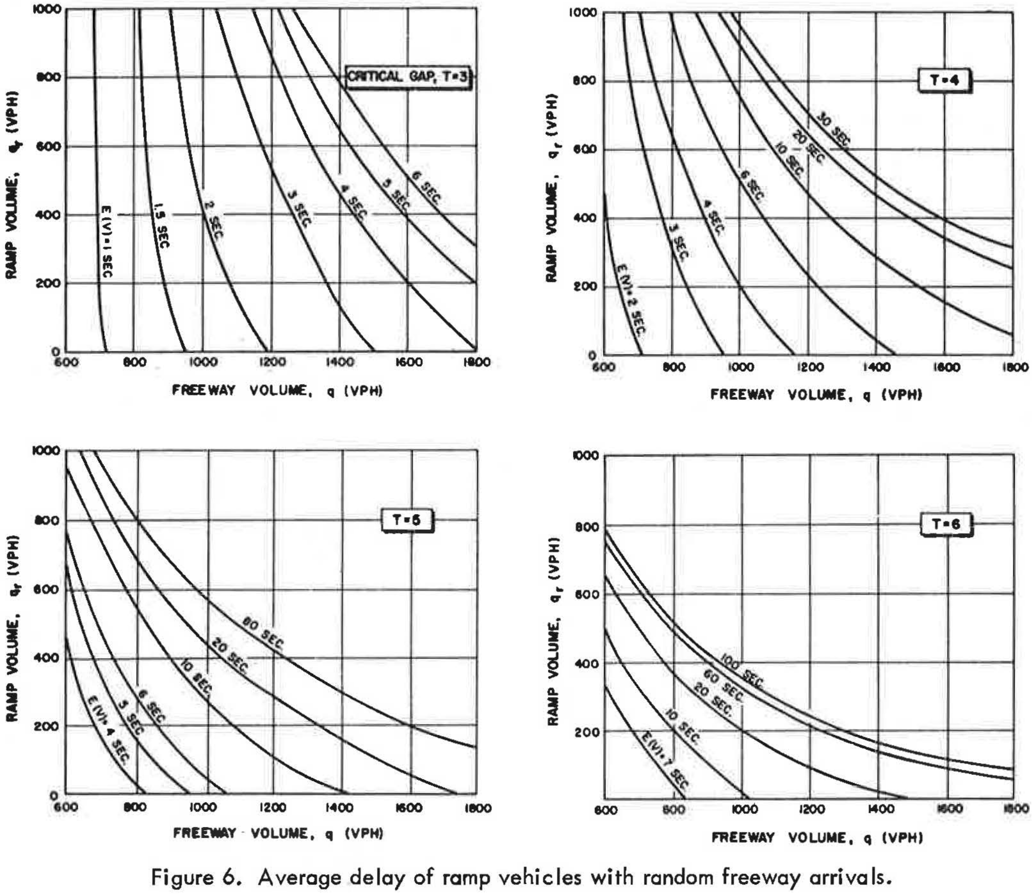

Figure 6. Average delay of ramp vehicles with random freeway arrivals.

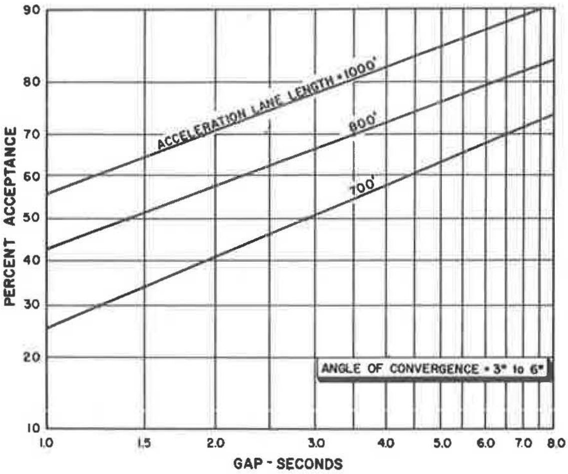

Figure 7. Effect of acceleration lane length on gap acceptance characteristics.

delineated entrance ramp that would eliminate any confusion in distinguishing between the entrance ramp elements and the main freeway lanes.

Translated into geometric variables, the critical gap depends primarily on the length of acceleration lane, L, and the angle of entry, θ. This effect was clearly evidenced by the study data, as demonstrated in Figures 7 and 8. Figure 7 shows the effect of the length of acceleration on the gap acceptance behavior of drivers for ramps having convergence angles from 3 to 6 degrees. The effect of the angle of convergence on percent acceptance is shown in Figure 8 for ramps having acceleration lanes between 650 and 800 ft. It can be seen that 50 percent of the drivers accepted gaps less than 1.5 seconds at the entrance ramp with a 3-degree angle of convergence whereas the 50 percent percentile gap is 3.5 seconds for an 11-degree angle (8).

To determine the effects of acceleration lane length and angle of convergence on the critical gap and to develop a relationship between the geometric variables and the gap acceptance characteristic of an entrance ramp, two sets of regression analyses were performed. Using as input data the gap acceptance characteristic developed from each data film (8), regression equations were found for the 50 percentile or critical gap and for the slope of the gap acceptance line, using a stepdown procedure.

The critical gap is given by

$$T = 5.547 + 0.828\theta - 1.043L + 0.045L^2 - 0.042\theta^2 - 0.874S$$

where

θ = angle of convergence in degrees;
L = length of acceleration lane in stations; and
S = shape factor
= 1 for taper type
= 0 for parallel type.

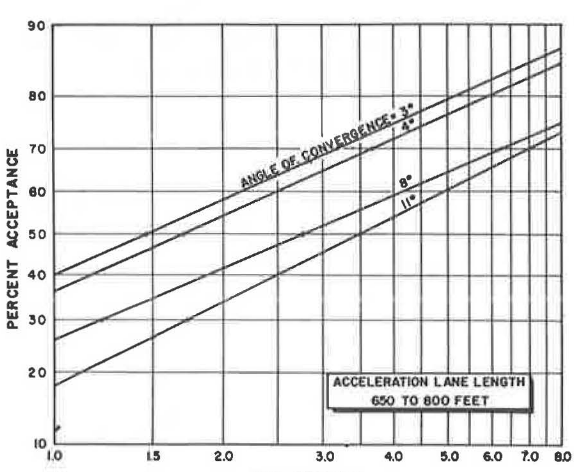

Figure 8. Effect of convergence angle on gap acceptance characteristics.

From the equation, it can be seen that increasing the angle increases the critical gap while increasing the length decreases the critical gap. As expected, then, a lower angle of convergence and a longer acceleration lane are desirable in the design of the ramp junction. Although this is not new, the significance of the above equation is that it quantifies these effects so that the designer can determine what is gained or lost by varying

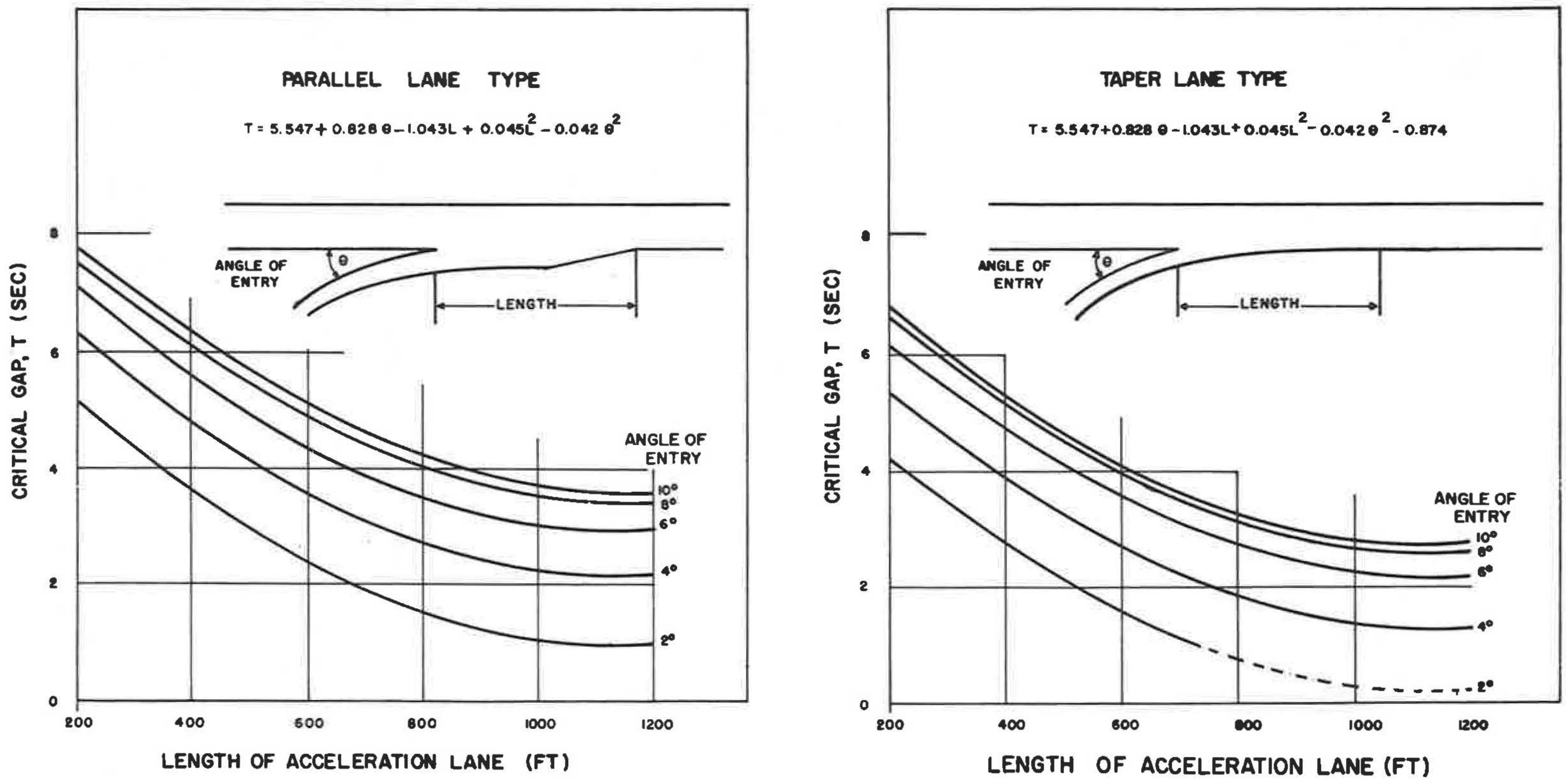

Figure 9. Critical gap, T, based on entrance ramp geometrics.

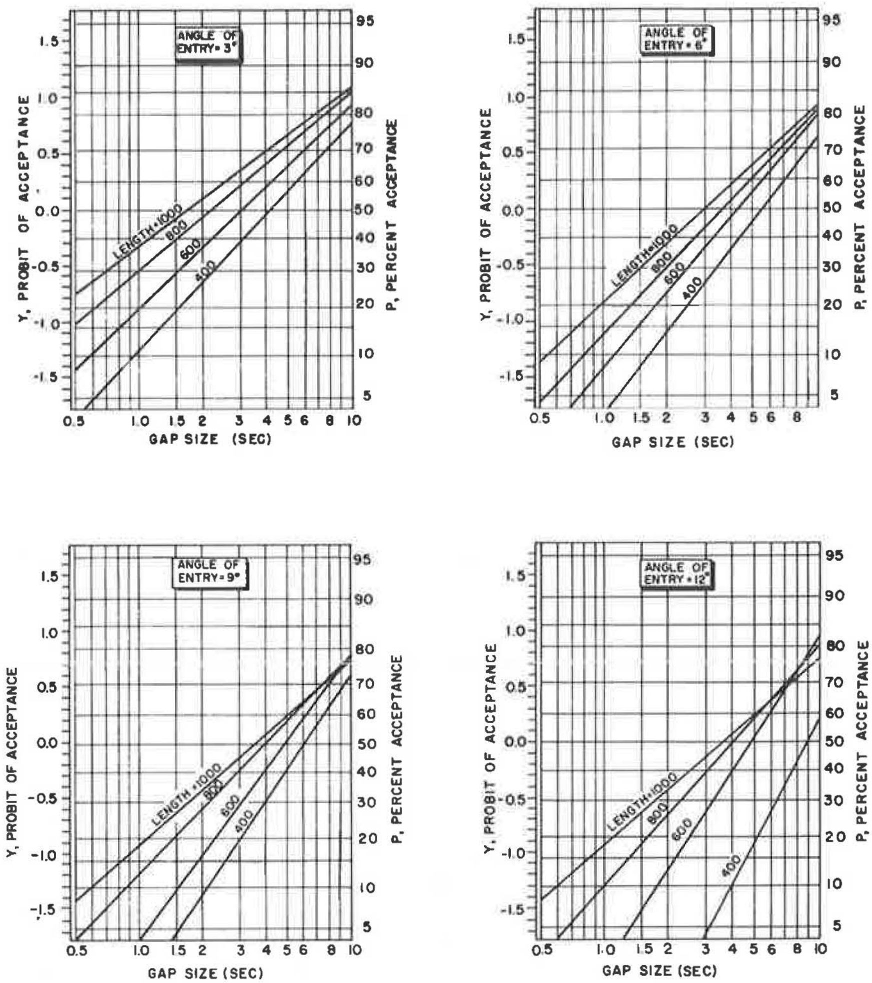

Figure 10. Gap acceptance characteristics as related to entrance ramp geometrics.

θ and L. Curves for estimating the critical gap from the length of acceleration lane and angle of entry are shown in Figure 9.

Another interesting aspect of this analysis is that it shows, for the first time in the literature, a difference between a taper type and a parallel lane type of acceleration lane—a topic that has given rise to considerable controversy. According to the analysis, which is based on observations of the operation of 13 taper type and 16 parallel lane type acceleration lanes, a tapered entrance terminal will, on the average, have a critical gap that is about 0.9 second smaller than that of a parallel lane type acceleration lane with the same length and the same angle of convergence. This finding is not to be construed as an unconditional endorsement of the taper type junction. Other factors such as grade, ramp length, and curvature and environment also play an important part. The limitations of the data and the foregoing analysis should be kept in mind. Nonetheless, based on the study data, it appears that under identical conditions the taper type acceleration lane has, on the average, a more favorable gap acceptance characteristic than the parallel lane type.

The stepdown regression analysis, with the slope of the gap acceptance line as the dependent variable, yielded the following equation:

$$B_1 = 1.394 + 0.289\theta - 0.027L\theta$$

where

θ = angle of convergence in degrees;

L = length of acceleration lane in stations;

B_1 = slope of the gap acceptance line as expressed by $Y = A + B_1X$, Y being the probit and X the logarithm of the gap size.

An increase in the angle increases the slope of the gap acceptance line and thus decreases the variance of the critical gap distribution while the acceleration lane length has the opposite effect for a fixed angle of entry. Note that the shape of the acceleration lane does not affect the variance of the critical gap distribution.

The gap acceptance characteristics based on the two regression equations are shown in Figure 10 for different angles of convergence and different lengths of parallel acceleration lane.

APPLICATION TO DESIGN

Freeway Ramp Design Procedure

There are three basic procedures employed in checking capacity for the design of entrance ramps. One method is based on preventing the total freeway volume upstream of the ramp plus the entrance ramp volume from exceeding the capacity of a downstream bottleneck. A second method takes into consideration the distribution of freeway volumes per lane and then limits the ramp volume to the merging capacity less the upstream volume in the outside lane. The third method discussed in this report states that the ramp capacity is limited by the number of gaps in the shoulder lane that are greater than the critical gap for acceptance.

Figure 11, which is a modification of Figure 3, can be useful in the implementation of all three approaches. Thus, if a ramp on a new facility is of a high-type geometric design guaranteeing a low critical gap, methods 1 and 2 are applicable since the merging service volume will exceed any bottleneck service volume. However, if, due to the terrain, spacing of interchanges, or ramp configuration, some compromise in the geometric design of the ramp-freeway merging area is necessary, then the third method should be employed.

The effect of poor ramp geometrics is evident. Consider the differences in ramp service volumes for a shoulder-lane flow of 1200 vph (a = 3 from Figure 4) as the critical gap T increases from 3 to 4 seconds. From the lower left-hand graph in Figure 11, one sees that q_r drops from 480 vph to 160 vph. To have used some arbitrary merging service volume (say, 1800 vph) in the capacity check for this freeway would have been a dangerous oversimplification. Actually, the entrance ramp design capacity curves in Figure 11 or Figure 5 should be employed so that the individuality of each ramp junction is considered. The values of the critical gap, T, needed to enter the curves are, of course, obtained from Figure 9.

Freeway design is, as are most real-world phenomena, a series of compromises. Because of the spacing of interchanges on many urban freeways, the fulfillment of desirable entrance ramp design, desirable exit ramp design, and the provision for an adequate weaving section between them often offers a dilemma. The alternatives are (a) reduction in the standards of one or more of the features, (b) elimination of one of the features (such as one of the ramps), or (c) transferring the weaving from the freeway to the frontage road. These alternatives should be evaluated in terms of their cost and their effect on adjacent facilities such as adjacent interchanges, and cross-street signalization. The procedure described in this paper enables a designer to evaluate alternatives more rationally and, if compromise is needed, to select the element or location where it will be the least objectionable.

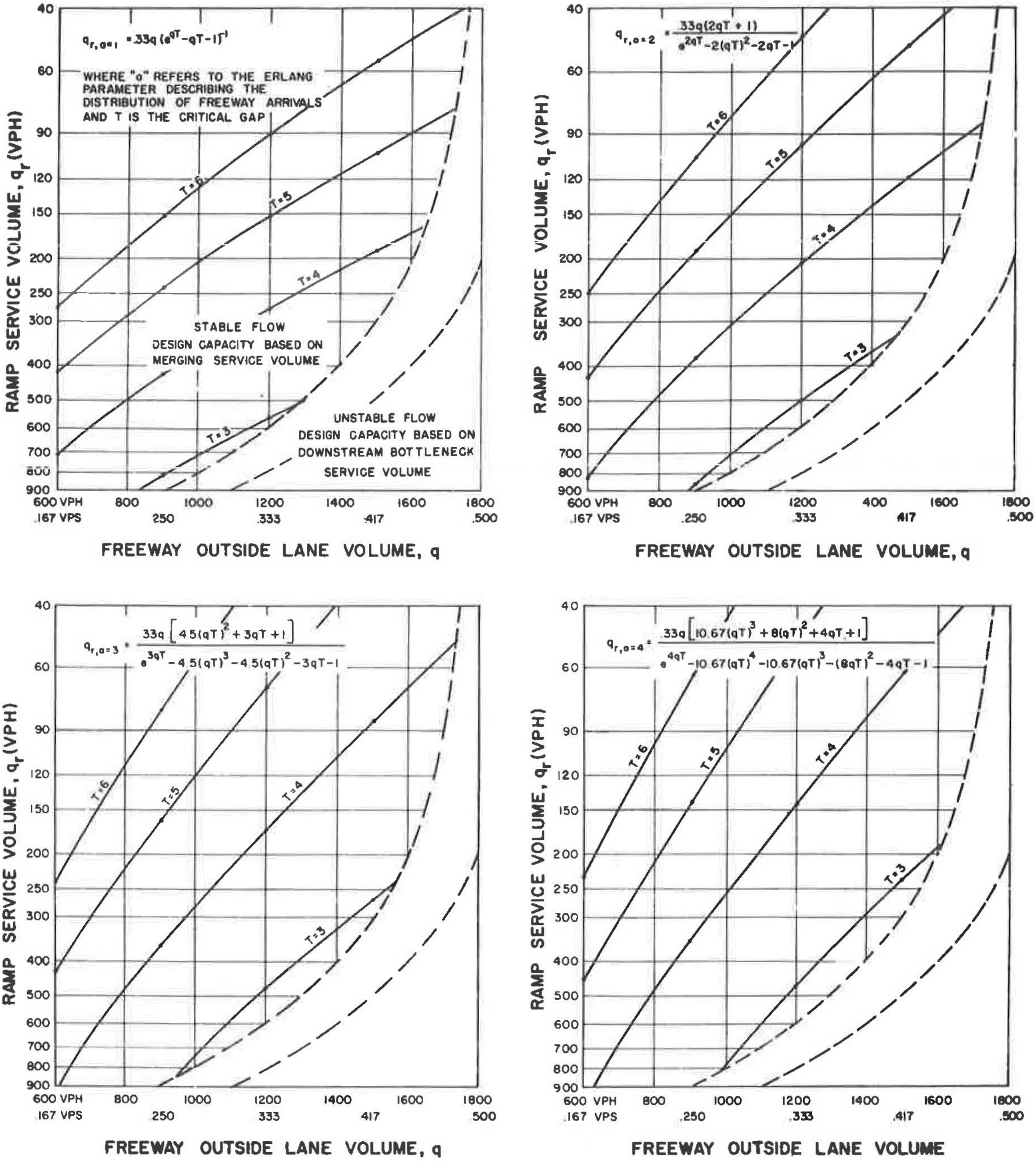

Figure 11. Entrance ramp design capacity curves.

APPLICATION TO CONTROL

Freeway Surveillance and Control

The term "surveillance" has developed in the highway terminology primarily in the last decade and denotes the observation of conditions in time and space. Initially, urban freeway surveillance was limited to moving police patrols. Recently, helicopters have been used for freeway surveillance in many metropolitan areas. Efficient operation of high-density freeways is, however, more than knowing the location of stranded vehicles or the qualitative description of the degree of congestion by high-flying disk jockeys. Television surveillance became an operational reality in the late 1950's both in the United States and Europe. The Port of New York Authority used closed-circuit television for monitoring traffic in the Hudson River tunnels and in Germany, a well-publicized TV system was developed to monitor traffic at a major complex urban intersection.

Experimentation with closed-circuit television as a freeway surveillance tool was initiated on a 3-mile section of the John C. Lodge Freeway in Detroit. This offered the opportunity of seeing a long area of highway in a short, almost instantaneous period of time made possible by spacing cameras along the freeway so that a complete picture could be obtained of the entire section of the roadway. The system was put into use in the summer of 1961.

A similar closed-circuit television system exists on a 6-mile section of the Gulf Freeway in Houston. It permits complete surveillance of the traffic flow as well as the expedient handling of accidents or stalled vehicles on the freeway. The television monitors are housed in a central control center, shown in Figure 12.

Making better use of traffic facilities has long been a basic concept of the traffic engineer. However, the installation of access controls on freeways to obtain better traffic flow was not originally conceived for these facilities. The rapid growth of traffic demand in our urban areas, coupled with the long-term construction requirements for building an extensive urban freeway system, has required the application of a control concept to freeway operation.

Evolution of Ramp Control Criteria

When demand exceeds or sometimes only approaches the capacity of a system, there is a self-aggravating deterioration of operation and build-up of congestion. In such cases, classical control systems are employed to either make the facility flexible enough to accommodate fluctuations in demand or to reduce the magnitude of the demand fluctuations. Freeway surveillance and control projects are necessarily limited to the latter. One approach, pioneered by the Detroit project, is to inform the motorist of traffic conditions by using lane controls and variable speed messages. A second and more positive approach is exercised at the point or points of ingress,

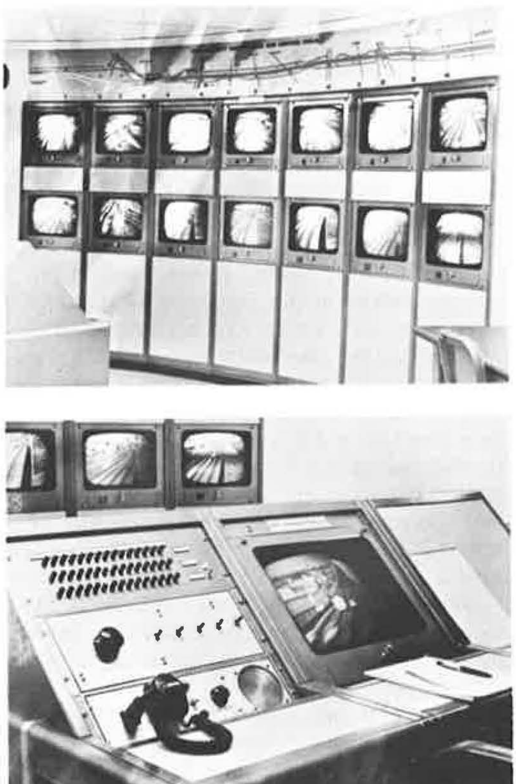

Figure 12. Monitor arrangement in central control center.

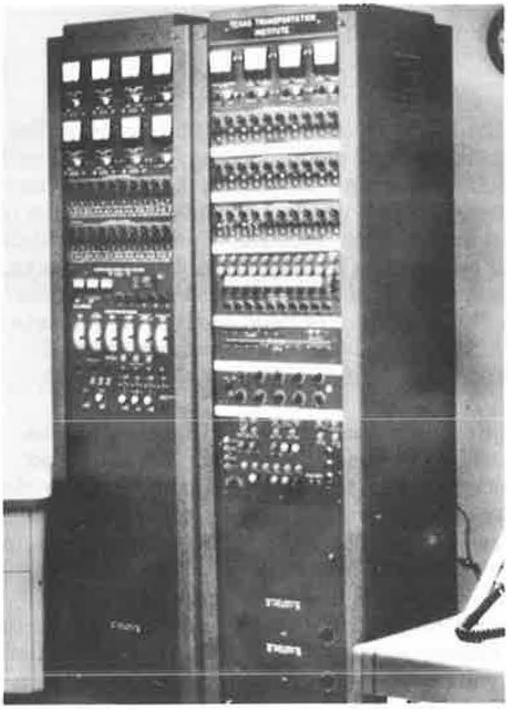

Figure 13. Automatic ramp controllers.

such as the entrance in the case of tunnel control or the on-ramps in the case of freeway control.

Metering, the process of controlling the amount of entering traffic, was developed by the Port of New York Authority. The first step was the identification of the bottleneck at the foot of the tunnel upgrade. Secondly, a mathematical model (15) was formulated to describe the behavior of vehicular traffic in the tunnel. The significant feature of the model was its prediction of shock waves upstream of the bottleneck. The remedy consisted of metering traffic at the entrance of the tunnel to prevent the development of instability by keeping traffic density below some critical value and by keeping the traffic demand below the bottleneck capacity.

Based on the success of metering in the tunnel, a similar control plan was formulated for the Eisenhower Expressway by the staff of the Chicago project. Two bottlenecks on the outbound facility were identified within the study area (16). The one farthest upstream is caused by a reduction in the number of lanes from four to three without a corresponding reduction in traffic demand. The second bottleneck, farther downstream and the last bottleneck on the outboard expressway, is caused by fairly heavy on-ramp traffic and is located at the top of an approximate 100-ft 3 percent upgrade.

Two metering techniques were developed. One technique used a point density or occupancy measurement on the freeway just upstream of the entrance ramp to be metered; the other used a volume measurement on the freeway about one-half mile in advance of the entrance ramp, and an exit ramp volume between the freeway volume measurement and the entrance ramp. After further study, the technique based on occupancy was selected in which a value of 15 percent occupancy on the center lane was used as a control parameter for initiating metering. From a relation established between the center lane occupancy and the maximum safe ramp volume, a metering rate was established for various levels of occupancy.

Figure 14. Ramp control signal.

Some researchers who followed the Chicago experiments were more impressed by the use of a freeway capacity-demand relationship as a control parameter for ramp metering. Wattleworth (17) has championed this "capacity-demand" criterion in which an individual ramp would be metered according to the difference between the upstream freeway demand and downstream freeway bottleneck capacity. He has also developed a linear programming model in which several entrance ramps in a freeway system would be metered so as to maximize the output of the system subject to constraints assuring that the demand will not exceed the total directional capacity at each freeway bottleneck (18).

In a paper presented in 1963, Drew (19) described a "moving queues" model based on coordinating ramp metering with the detection of acceptable gaps in the outside freeway lane. An acceptable gap is defined as one equal to or larger than the critical gap (that gap for which an equal percentage of ramp traffic will accept a smaller gap as will reject a larger one). Moving queues or platoons occur when the time headway or gap between successive vehicles is less than an arbitrary queuing headway. Since the queuing headway is taken as the critical gap, the number of ramp vehicles to be metered in some time-constant equals the number of moving queues detected. The average number of vehicles per moving queue, as the reciprocal of the probability of a gap larger than the critical gap, provides a rational index of freeway operation. The model has the flexibility of metering a single ramp vehicle per available acceptable gap on the freeway or metering ramp vehicles in bunches or platoons using a "bulk service" technique (12).

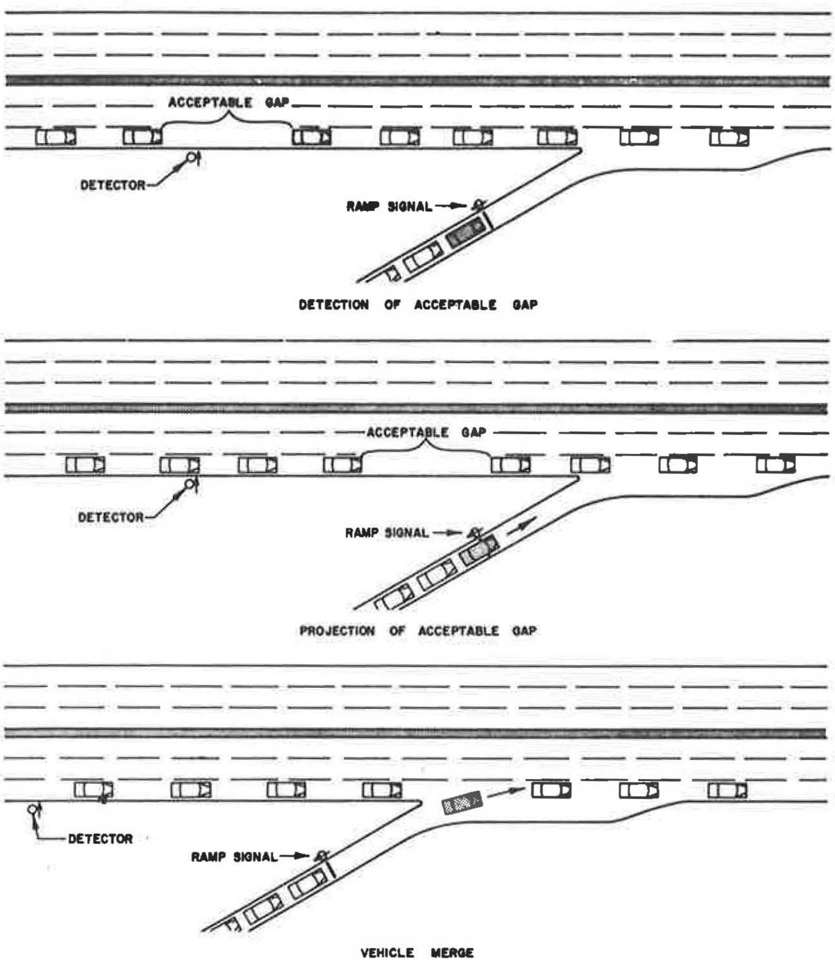

Figure 15. Illustration of gap acceptance mode of ramp control.

Automatic Ramp Control

Before an automatic ramp metering system is designed, its purposes and objectives should be considered. Assuming the proposed ramp metering system is both a research and an operational tool, it should involve the continuous sampling of basic traffic characteristics for interpretation by established parameters, in order to provide a quantitative knowledge of operating conditions necessary for immediate rational ramp control. In short: the system should be traffic-responsive, and it should be automatic.

Functional specifications were developed by the Texas Transportation Institute on a companion project sponsored by the Texas Highway Department and Bureau of Public Roads for the controllers shown in Figure 13. The controller on the right is called the "Gap Acceptance Mode" or Mode I. It detects gaps (or time spacing between vehicles) in the outside lane of the freeway upstream of an entrance ramp and evaluates the size of these gaps with regard to their ability to accommodate a vehicle entering from the ramp. When a desirable freeway gap is detected, it is projected downstream by means of a delay circuit to a point where a waiting vehicle on the entrance ramp can be merged into the gap. At this time, the signal on the ramp (Fig. 14) turns green and releases a ramp vehicle for a smooth merge into the freeway. The functional process followed by the controller is illustrated in Figure 15.

Operation of the Gap Acceptance Mode

The Gap Acceptance Merging Control Mode, designated Mode I, was installed in March 1966 on the Telephone Road inbound entrance ramp of the Gulf Freeway. The control of the signal is completely automatic. Loop detectors on both sides of the signal provide the calls for the green and red signals. Control is designed for either single vehicle or multi-vehicle entry. The detectors, speed and volume computers, and signal controller are rackmounted in the Surveillance Center. The closed-circuit television system in the Surveillance Center is used to observe the operation of the signal.

The control of the ramp signal is basically by the detection and projection of acceptable gaps. However, because of the nearby intersection, the length of queue waiting at the signal has a control function. As a safeguard against long delays of the signal due to slowdowns on the freeway lanes, the speed of traffic in the outside lane is a second basis for control. There is also a provision for keeping the ramp area from signal to the freeway clear. These functions are explained in the following paragraphs (20).

Gap Projection—A loop detector, placed about 950 ft upstream of the ramp nose (see Note 1 in Fig. 16), measures all gaps in the outside lane and allows the calculation of the speed of traffic flow (Note 2, Fig. 16). When a gap is detected that is equal to or greater than the designated acceptable gap size, it is projected in the controller at a rate defined by the speed in the outside lane. If a ramp vehicle is waiting at the ramp signal, a call for the green signal is made when the projected gap reaches that position in time, designated the decision point, at which the travel time of the gap to the merge area is the same as the anticipated travel time of the ramp vehicle from the signal to the merge area (Note 3, Fig. 16). However, the green signal will not be called if there is a ramp vehicle stopped over the merge detector (Note 6, Fig. 16).

If the gap is equal to or greater than the designated acceptable gap size for more than one vehicle, the controller holds the green signal until the gap passes the decision point (Note 7, Fig. 16).

Speed of Outside Lane Traffic—A loop detector placed at the nose of the entrance ramp measures the speed of traffic flow, which is used to select the size of the acceptable gap. When speeds in this area drop below a preset speed, a background cycle rate, set on a fixed rate control, is put into effect. The signal continues to release vehicles when acceptable gaps become available, but it also releases vehicles after a specified waiting time. The difference in this rate and the rate called by the queuing detector described in the next paragraph is that this rate is a minimum setting, and is called when the freeway is in a very slow and congested condition. This control overrides the queuing detector. The fixed rate setting is usually in the range of 150 to 200 vehicles per hour.

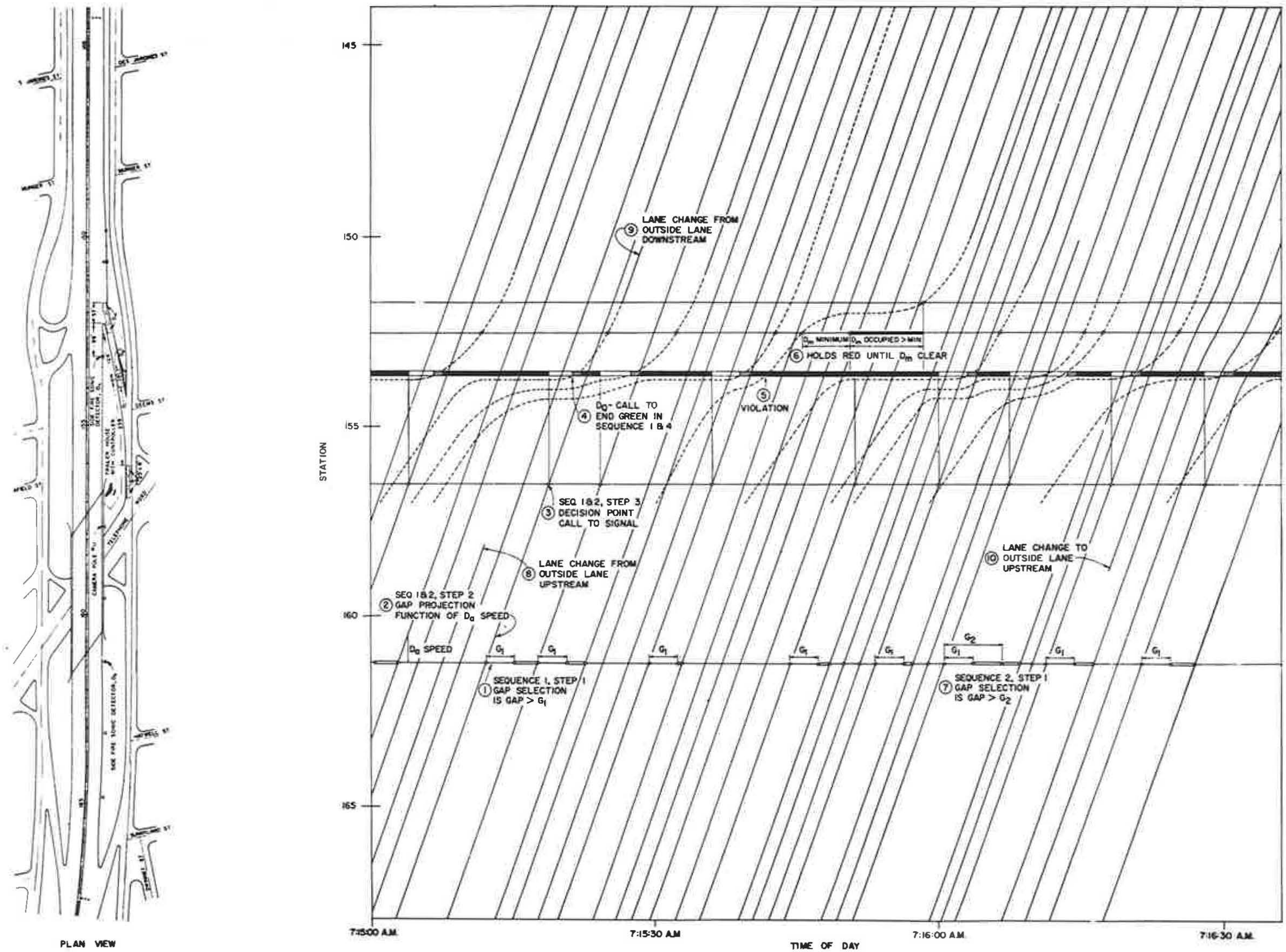

Figure 16. Time-space diagram illustrating operation of gap acceptance ramp metering mode.

Length of Queue—A loop detector is placed in the pavement of the left lane of the inbound frontage road near the Telephone Road intersection. If the queue at the ramp signal is greater than 14 or 15 vehicles so that this loop becomes occupied for longer than a preset time interval, a background cycle rate, set on a guaranteed rate control, is put into effect. This rate stays in effect only as long as the queuing detector is timed out. The guaranteed rate setting is usually of the order of 500 to 600 vehicles per hour.

Occupancy of the Merge Area—A loop detector is placed in the pavement of the ramp just upstream of the merging area. All vehicles entering the freeway from the ramp will actuate the detector. If a vehicle stops on the ramp in this area, blocking the entrance to the freeway so that the detector times out a preset interval, the controller will hold the signal in red until the detector is cleared (Note 6, Fig. 16).

Ramp Metering vs Merging Control

The interest of research lies in the subtle blending of theory and experiment. Scientific theories excite the curiosity; they are always useful, and occasionally they may be even beautiful. Based on theory, we predict as precisely as possible what should happen in some new experiment. While carrying out the appropriate test, we are hoping that the theory will work, and our moment of triumph occurs when our theory has predicted some new phenomenon accurately for the first time. On the other hand, if the theory is not ours but a rival theory, there is, added to our own curiosity in setting up the experiments to test it, a sense of rivalry. We now plan and carry out the experiments hoping to disprove this theory; similarly, the rival researcher is planning to try to disprove our theory. Attempting to disprove theories in this way is a very important part of scientific endeavor that can be compared to the role of the opposition party in government. This common interest of many researchers in the same phenomenon causes some new theories to be refuted and others, after repeated testing, to be accepted, and as such is necessary to the growth of science.

The significance of the Gap Acceptance Merging Control Mode lies in its conceptual appeal. Note the use of the term "merging control" rather than "ramp control" in describing the mode. The Gap Acceptance Mode is the only metering system that attempts to aid the ramp driver in the merging maneuver. This is important and shall be explained in more detail.

When the volume of traffic on a freeway begins to approach capacity, the merging driver is placed in an extremely difficult position. The number of acceptable gaps in the freeway stream decrease sharply as the freeway volume increases. At these higher volumes, the merging driver cannot always defer his decision to merge until he is on the acceleration lane. Instead he must detect the location of gaps in the oncoming stream before he reaches the acceleration lane. Operating in this manner, he must project the progress of a gap onto the acceleration lane in order to decide whether or not it will be available to him. This, in turn, requires that he estimate his own speed and acceleration as well as the speed and size of the gap in order to decide whether there will be sufficient space for the merging maneuver to be completed successfully within the limit of the acceleration lane.

Michaels and Weingarten (21) do not think it is possible for the driver under these circumstances to reliably solve the appropriate equations of motion. They state:

> It is obvious that as the mainstream volume approaches capacity, the merging driver's task becomes for all practical purposes impossible. Thus, effective ramp metering will require the equations of motion to be solved automatically whenever a vehicle enters a ramp. Mathematically, the problem is quite simple, requiring a knowledge of the location of gaps and their speed. Knowing something about the accelerating capability of the ramp vehicle and the length of the ramp and acceleration lane, a perfectly determinate solution is possible. Instrumentation to carry out these operations is well within the state of the art of existing electronic technology.

The gap-oriented system installed at the Telephone Road interchange locates freeway gaps and their speeds, compares these gaps to a "critical gap," takes into account the accelerating capability of the ramp vehicles and the length of the ramp and acceleration lane, and solves the equations of motion automatically before the metering signal is actuated to allow a vehicle, or vehicles, to make a smooth merge. In addition to the increased efficiency so obtained, other factors such as safety and higher ramp capacity are improved for a comparatively low installation cost.

Considering safety, any speed differential at a point in the traffic stream in either a longitudinal or transverse direction is dangerous. A vehicle that stops in a traveled lane is in particular danger; it is a safety hazard to the remaining traffic, to its driver, and to its occupants. This is indicated by the high percentage of accidents of the rear-end type that occur at induced stop and yield locations such as the freeway merging area. The Gap Acceptance Mode virtually eliminates ramp vehicles stopped in the merging area, thereby contributing greatly to safe operation. In addition, the system affords the opportunity for increased ramp capacity over other metering models. In systems that meter ramp vehicles one at a time, the ramp capacity is obviously a function of the ramp cycle length. Since it takes about 4 seconds to go through the ramp signal cycle, the maximum metering is at a rate of one vehicle every 4 seconds or 900 vph. The Mode I system can meter at a faster rate because it has the flexibility to meter more than one ramp vehicle whenever large freeway gaps are detected.

In conclusion, the Gap Acceptance Mode provides the merging driver with the necessary information to know that a sufficient gap is available. Second, because of its nature, it is also a metering system. Such a dynamic merge-aiding and metering technique appears to be a very attractive and inexpensive way of maintaining high efficiency of flow on a freeway and at the same time of obtaining maximum ramp capacity and merging safety.

Integrated Freeway Control

With the exception of the Gap Acceptance Merging Control Mode, all the ramp metering techniques in use today may be classified as macroscopic in nature. For example, in the capacity-demand criterion of ramp metering, a ramp is metered according to the difference between the upstream freeway demand and downstream bottleneck capacity. In other words, steady-state stability is maintained as long as the demand-capacity ratio is less than unity. On the other hand, the merging control criterion is microscopic in nature since it considers each freeway gap and each ramp vehicle individually.

The graphs in Figure 11 point out the differences in the macroscopic philosophies of ramp metering and the microscopic merging control approach. In the macroscopic approach, metering would be based on one of the dashed lines (one representing the boundary between stable and unstable flow and the other representing capacity) regardless of the ramp geometrics or critical gap. This means that for all conditions except those described as unstable flow on the graph, vehicles would be metered at a faster rate than the service rate at the merging area, as dictated by the available critical gaps, forcing drivers to either accept smaller gaps than the critical gap or become part of a steadily growing queue at the merging area.

This figure also illustrates the need for a ramp control technique combining both the macroscopic and microscopic approaches. For conditions described on the graph by "unstable flow," the ramp geometrics do not govern and hence the macroscopic approach based on the downstream bottleneck service volume applies. However, to the left of the 1800-vph line dividing stable and unstable flow, the critical gap governs, since the merging service volume is less than the bottleneck service volume.

Looking ahead, it is well known that optimization of a part of a system or subsystem does not necessarily lead to the optimum solution for the entire system. Similarly, optimizing the operation of a single merging control system may not necessarily lead to the optimization of the overall system. The entrance ramp control curves in Figure 11 afford the flexibility of controlling all the ramps in a freeway system according to either the individual merging areas or the downstream bottlenecks.

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Criteria To Be Used in Developing Warrants for Interchanges on Rural Expressways

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•INCREASING travel demands between cities have prompted highway officials to plan for the replacement of many antiquated two-lane highways with high-type multi-lane divided expressways. [The term "expressways" as used here is defined as "divided arterial highways for through traffic with full or partial control of access" (1).] It is apparent that the Interstate System alone will not provide adequate service for all high-volume traffic corridors. Many state highway departments have recommended that extensive expressway systems be built to connect intermediate-sized cities, thus alleviating conditions involving inadequate capacity, high accident rates, and intolerable driver delay and inconvenience.

Proposed design standards for the supplemental expressways specify control of direct access and high design speeds. One vexing problem to be decided on many of the proposed expressways is the treatment of intersections between the expressways and other highways. In most cases, it is considerably less expensive to build at-grade intersections rather than grade-separated interchanges because costly grading, bridging, and ramp construction can be avoided. However, if interchanges are constructed, many stops and slowdowns can be eliminated. These maneuvers are costly to motorists in both time and operating expense.

To facilitate decision-making, established warrants are needed for use in selecting proper intersection treatment at various traffic conditions. Present warrants lack a quantitative basis from which analytical comparisons can be made of consequences resulting from the construction of either an at-grade intersection or an interchange. With a sound knowledge of these consequences, the decision to designate an expressway as a freeway can be objectively made.

SELECTION OF CRITERIA

Three basic criteria—level of service, safety, and economics—should be taken into account in deciding the proper treatment of intersections. The degree of driver inconvenience and delay is an indicator of the level of service provided by a highway facility. One of the most common ways of measuring the relative level of service between two alternative design treatments is to determine the savings in travel time.

Public safety must be a major concern in planning highway systems. It is not yet possible to estimate reliably the number of accidents that can be saved by constructing an interchange rather than an at-grade intersection. Much more research is needed to relate accident experience with specific design features.

Another fundamental criterion is the comparison of user benefits realized by providing an interchange rather than an at-grade intersection with the extra cost involved in constructing and maintaining the interchange. In this study, the ratio of benefits to costs for a diamond interchange vs a signalized intersection is presented as a function of traffic conditions.

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DESCRIPTION OF MODELS

The movements of vehicles traversing both the at-grade signalized intersection and the diamond interchange were simulated so that the complex interaction between vehicles could be quantitatively measured. Operating costs were determined for each vehicle by applying unit costs based on the type of maneuvers undertaken by the vehicle.

Signalized At-Grade Intersection

The computer program developed by Gerlough and Wagner (6) was chosen to simulate traffic moving through the at-grade intersection because of its inherent realism and flexibility. With this program, all input parameters could be controlled to determine the effect of changing traffic volumes.

During the simulation process, a record of the behavior of each vehicle as it traversed the intersection was recorded. Data on time, position, acceleration rates, and speed were punched on cards whenever a vehicle entered the system, changed lanes, leveled-off speed after acceleration or deceleration, or left the system. This output was used as input to the vehicle operating cost program. In this computer program, consecutive cards for the same vehicle were compared to determine whether the vehicle had been traveling at uniform speed, idling, or changing speed. If the speeds were the same, a cost for traveling at a uniform speed was calculated by multiplying the applicable unit cost for that speed by the distance traveled. Different unit costs were used for passenger vehicles and trucks.

If both speeds were zero, the cost of idling was calculated by multiplying the unit idling cost by the difference between the times on the two cards. If the speed had increased or decreased, the cost of either accelerating or decelerating was calculated by adding the excess cost of changing speeds to the cost of operating at the higher speed. The excess cost of changing speeds was a function of both the initial and final speeds. Whenever a vehicle left the system, the operating costs for that vehicle were totaled and the total elapsed time was calculated. These costs and times were added to costs and times of other vehicles making the same movement. At the end of the run, average costs and travel times for each movement were determined and printed on a summary report. The traffic volumes simulated are given in Table 1.

The mean values obtained in each of the 19 simulation runs were used to develop regression equations, which yielded either unit operating costs or mean travel times for each movement as a function of hourly arterial and expressway traffic volumes. Passenger vehicles and trucks were treated separately in the regression equations. Operating costs and total travel times during an average day were determined by applying the regression equations to the traffic volumes occurring during each hour of the day and summing over 24 hours.

Diamond Interchange

Behavior patterns and travel times of vehicles traversing the diamond interchange were obtained by simulating the operation of cars and trucks having mean speed, acceleration rates, and deceleration rates. Speed-time graphs were used to analyze both

TABLE 1

AT-GRADE INTERSECTION SIMULATION RUNS

Arterial Volume (vph)	Expressway Volume (vph)				
	200	500	800	1,100	1,500
100	x	x	x	x	x
250		x	x	x	x
400		x	x	x	x
600			x	x	x
800			x	x	x

car and truck maneuvers for each turning movement. Stopped-time delay at the stop sign located on the ramps at the arterial highway was derived from data presented by Kell (10). The operating costs of vehicles traversing the interchange were calculated using the same procedures employed in the at-grade vehicle operating cost program.

INPUT PARAMETERS TO MODELS

Geometric Configuration

The boundaries of the study were chosen as 2,000 ft upstream on each approach and 3,000 ft downstream on each exit. The geometric plans for the at-grade intersection and the diamond interchange are shown in Figure 1. Outside the intersection area, the expressway is a four-lane divided access-controlled highway. The arterial highway has been widened to accommodate two approach lanes near the intersection to provide

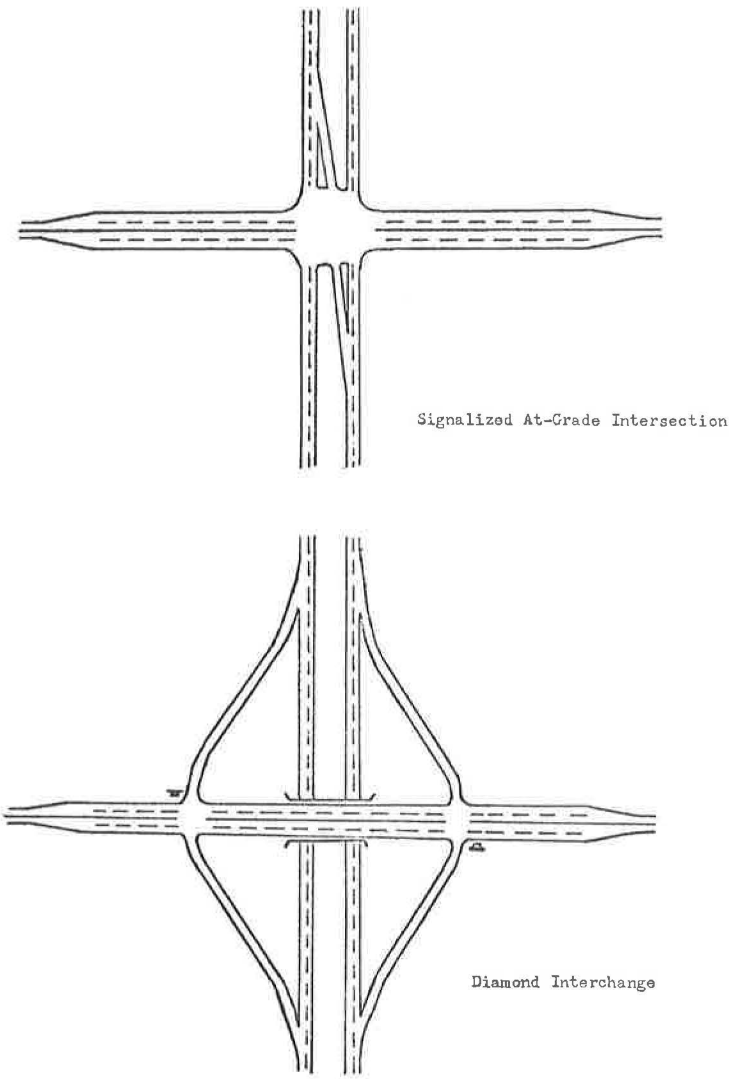

Figure 1. Geometric plans of the at-grade intersection and the diamond interchange.

additional capacity. Left-turn lanes on the expressway were introduced on the at-grade intersection mainly as a safety feature. A high type of design was selected for the at-grade intersection so that the results of the study would be conservative. Therefore, the actual benefit at a lower type of design would be even greater than that determined by this study.

The interchange ramps were designed for speeds of 40 mph. Adequate sight distance for turning traffic was provided by placing ramp terminals 570 ft from the center of the bridge.

Signal Timing

A two-phase pretimed signal was used to control the traffic entering the at-grade intersection. The majority of rural intersections are controlled by full vehicle-actuated signals. Because this type of controller logic was not written for the at-grade simulation program, a substitute controller was sought that would replicate the vehicular delay encountered at an actuated signal. Several researchers (2, 6) have found that if a pretimed signal is set according to Webster's (16) optimum timing formula, both stopped-time and total delay are very close to and sometimes even below that found at intersections controlled by traffic-actuated signals.

A cycle length was determined for each hourly traffic volume combination using Webster's formula. Initial starting delay was set at 3.5 seconds and the mean saturation flow used was one vehicle every 2.2 seconds. The computed cycle lengths agreed very closely with the optimum cycle lengths determined by Kell (10). Yellow times of 5 seconds were calculated using Olson and Rothery's (13) formula. Green time was allocated to the two phases according to the distribution of the critical lane volumes on the expressway and arterial.

TABLE 2

DRIVER CHARACTERISTICS USED IN THE SIMULATION MODELS

Driver Characteristic	Value Used	Source
Mean desired speed		Iowa Highway Comm. (9)
Autos		
Expressway	60 mph	
Arterial highway	55 mph	
Trucks		
Expressway	53 mph	
Arterial highway	48 mph	
Standard deviation of speed		Oppenlander (14)
Expressway	9.2 mph	
Arterial highway	8.2 mph	
Normal acceleration rates		
Autos		Field Study
0-10 mph	7.0 ft/sec^2	
10-20 mph	6.1 ft/sec^2	
20-30 mph	3.6 ft/sec^2	
30-40 mph	2.8 ft/sec^2	
40-50 mph	2.0 ft/sec^2	
50 mph and above	1.4 ft/sec^2	
Trucks		Deen (3)
0-10 mph	4.8 ft/sec^2	
10-20 mph	2.5 ft/sec^2	
20-30 mph	1.6 ft/sec^2	
30-40 mph	0.85 ft/sec^2	
40-50 mph	0.8 ft/sec^2	
50 mph and above	0.8 ft/sec^2	
Normal deceleration rates	-4.4 ft/sec^2	Ohio State Univ. (12)
Standard deviation of deceleration	0.7 ft/sec^2	Ohio State Univ. (12)
Maximum deceleration rate	-15 ft/sec^2	Norman (11)
Characteristic speed		Edie et al (4)
Expressway	32.5 ft/sec	
Arterial highway	29.8 ft/sec	
Arrival distribution	shifted exponential	
Driver reaction time	1.0 sec	Forbes et al (5)
Probability of stopping on yellow	varies	Gerlough and Wagner (6)
Probability of changing lanes	varies	Gerlough and Wagner (6)
Probability of gap acceptance by left turners	varies	Gerlough and Wagner (6)

Driver Characteristics

The driver characteristics used in the simulation models are given in Table 2.

Basic Traffic Parameters

During each simulation run, left and right turning movements were each set at 20 percent of the minor approach volume. Truck volumes were assumed to be 20 percent of the total volume on the arterial highway and 28 percent of the total volume on the expressway. Lane distribution of approaching straight-through traffic was obtained by field observations. The variation of traffic assumed to occur during each hour of an average day is given in Table 3.

The values assumed for the basic parameters are typical for the type of facilities being studied (7). Other values were also investigated to determine the sensitivity of the final results to these parameters.

COSTS

Unit Operating Costs

The unit operating costs used were those assembled by Winfrey (17). Costs included were for fuel (including 10 cents tax), tires, engine oil, vehicle maintenance, and vehicle depreciation. Winfrey's tables represented the findings of many recent studies concerning fuel and oil consumption and tire wear. Unit operating cost tables were presented for three different types of maneuvers: (a) traveling at uniform speed, (b) idling, and (c) changing speed. Winfrey presented tables for five types of vehicles. The tables for passenger vehicles and pickup trucks were weighted (93 percent passenger vehicles and 7 percent pickup trucks) in order to arrive at operating cost tables for a combination passenger car-pickup truck since these vehicles have similar operating characteristics. Likewise, cost tables for single unit and tractor-trailer semi-trailer trucks were weighted by the following distribution to arrive at cost tables for a typical truck: single unit trucks, 47 percent; 2-S2 TTST trucks, 31 percent; and 3-S2 TTST trucks, 22 percent. These distributions are representative for many types of arterial highways (7).

Time Costs

The benefit obtained by providing an interchange depends to a great extent on the value given to time. The proper value to use is controversial. Many authorities suggest values near $1.50 per hour. One question that might indicate that this value is too high for this analysis is: Do the small increments of time saved by each individual by providing an interchange actually bring an overall economic gain? Because of this controversy, values of both zero and $1.50 per hour will be considered.

TABLE 3

TYPICAL HOURLY TRAFFIC VARIATION ASSUMED TO OCCUR DURING AN AVERAGE DAY

Hour Period	Percentage	
	a.m.	p.m.
12- 1	1.4	5.6
1- 2	1.2	5.9
2- 3	1.2	6.2
3- 4	1.3	6.9
4- 5	2.0	6.6
5- 6	3.4	5.8
6- 7	4.8	4.9
7- 8	5.3	4.1
8- 9	5.5	3.4
9-10	5.5	3.2
10-11	5.3	2.8
11-12	5.1	2.6

Accident Costs

A reduction of accidents resulting from the provision of an interchange will provide an economic benefit. These benefits should be included in the economic analysis. Measures of accident costs have always been difficult to make. The costs used in this study were those determined by the Illinois Division of Highways (8). Only direct costs were considered. The distribution of the accident severity at rural signalized intersections was also determined by the Illinois Division of Highways (8). The same distribution

TABLE 4

AVERAGE COST PER ACCIDENT AND FREQUENCY OF ACCIDENT TYPE

Accident Type	Cost Per Accident	Frequency	
		Number	Percent
Fatality	$7,272	13	0.2
Personal injury	1,780	1,958	26.8
Property damage only	186	5,322	73.0
		7,293	100.0

was assumed for the diamond interchange. An average cost of $626 per accident was obtained by weighting the cost of each type of accident by its frequency. Frequency distributions and costs are given in Table 4.

Maintenance Costs

Few studies have provided detailed figures on maintenance costs. By using a rough figure of $5,000 per mile per year, it was estimated that maintenance costs would be $4,300 per year higher on the diamond interchange than on the at-grade intersection.

Construction Costs

The cost of providing either an at-grade intersection or a diamond interchange can vary greatly depending on many variables. Typical costs of high-type signalized intersections range from $200,000 to $350,000 while typical costs for diamond interchanges range from $480,000 to over $840,000. Because of these varying costs, three cost differences—$280,000, $400,000, and $680,000—will be compared with the benefit obtained by providing the diamond interchange.

An amortization period of 20 years was assumed. Because the extra money spent on the interchange could be invested in other worthwhile investments, an interest rate should be considered. Interest rates of zero and 7 percent will be used to determine the sensitivity of the results to this variable.

RUNNING THE AT-GRADE SIMULATION PROGRAM

By analyzing the variability of vehicle travel times, it was determined that a minimum sample size of 20 vehicles for each movement would be necessary. With this sample size, the sample mean would be within 7.5 percent of the true mean at the 95 percent confidence level. The amount of real time simulated during each run was established to provide at least this number of vehicles in each turning movement. Altogether, the behavior of about 7,700 vehicles was simulated in the 19 at-grade simulation runs.

RESULTS

Level of Service

The level of service provided by alternative highway facilities should be a major consideration in deciding which alternative should be built. There are two questions that should be asked when deciding between alternatives:

1. Does the level of service provided on each alternative meet the minimum acceptable standards set by the decision-maker?
2. Is the level of service provided by one alternative facility significantly superior to that provided by the other?

The level of service provided to drivers making each movement should be investigated to insure that minimum acceptable service is being provided to all movements

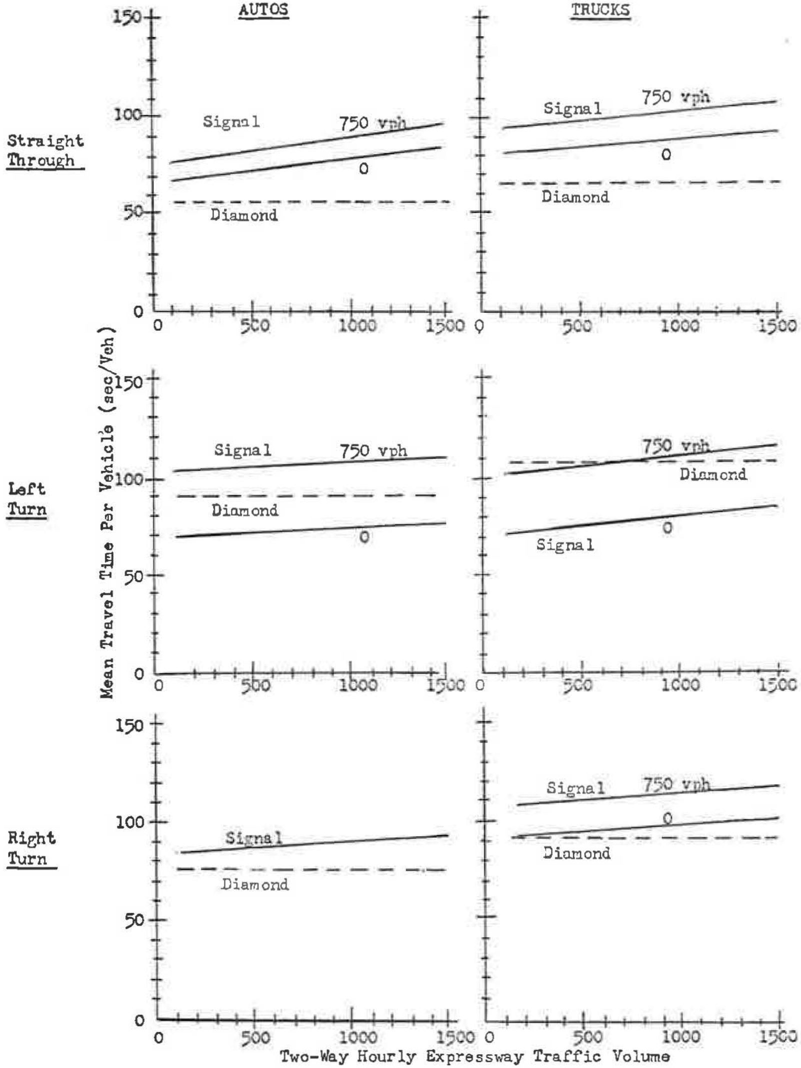

Figure 2. Mean travel time for vehicles approaching on the expressway.

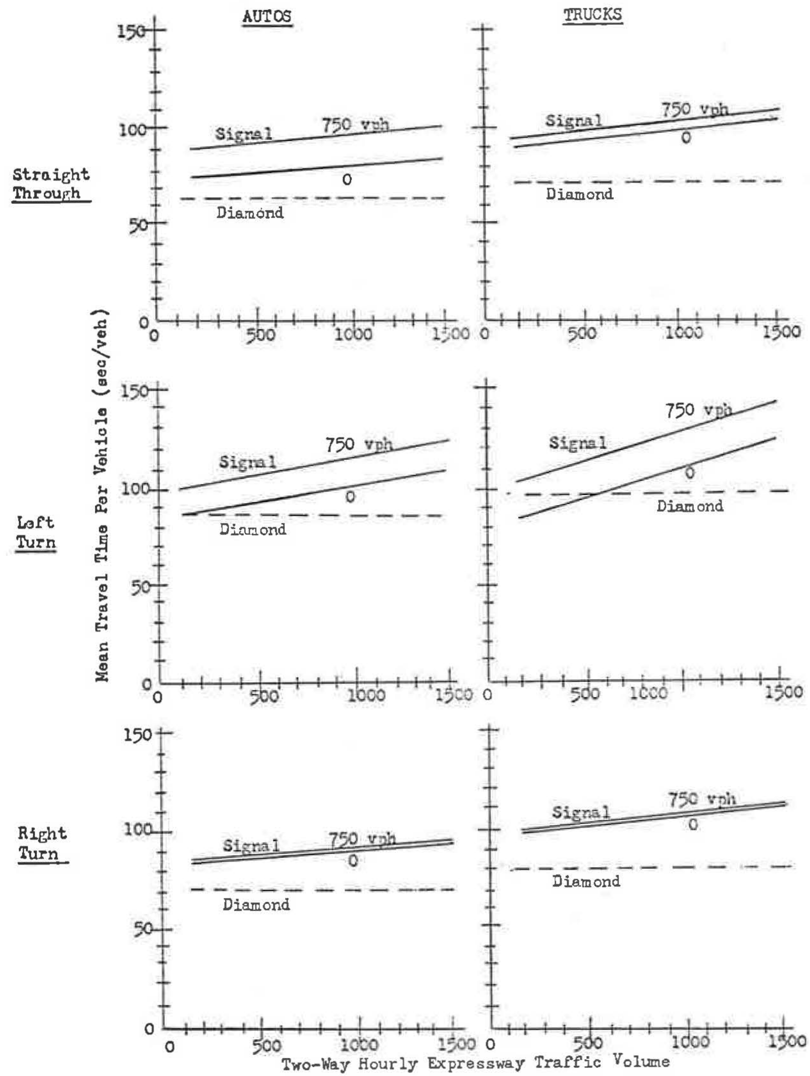

Figure 3. Mean travel time for vehicle approaching on the arterial highway.

during the design hour. Also, a particular movement may be of special concern to the decision-maker. For example, in an effort to provide a continuous high level of service on the expressway, the decision-maker might give priority to the alternative that provides the higher level of service to the drivers desiring to proceed straight through on the expressway.

Total travel time was the primary index of the relative service levels measured in this study. Comparisons of total travel time incurred by vehicles making each movement are shown in Figure 2 for the expressway and Figure 3 for the arterial. The travel times at the diamond interchange were always lower (about 50 percent lower at high traffic volume combinations) than travel times at the signalized intersection. However, because of the adverse travel for vehicles turning left from the expressway, travel times for this movement were greater on the diamond interchange than on the signalized intersection at low arterial highway traffic volumes.

Economic Analysis

Daily user benefits were determined for the 17 volume combinations given in Table 5 by summing every 24 hours the product of the unit costs of travel times and the traffic volume occurring during each hour of the day. Benefits for other volume combinations can be interpolated from the results presented. Values below 2,000 vehicles per day on the arterial highway were not considered because a signal would not be warranted at lower volumes under existing signal warrants (15).

Travel Time

A comparison of travel times incurred by the users of the signalized intersection and the diamond interchange is given in Table 6. While the total annual savings in travel time is quite substantial, the savings per vehicle average only about 20 seconds at the highest volume combination considered. However, at $1.50 per hour, this time savings results in a benefit of over $65,000 per year.

Operating Costs

The comparison of annual vehicle operating cost for the interchange and signalized intersection is given in Table 7. As can be seen, the savings are substantial at all volume combinations considered, ranging from $59,000 to about $318,000. This savings appears to be significant, since it is between 24 and 40 percent of the operating cost at the signalized intersection.

Accident Savings

A comparison of accident experience at the two alternative types of facilities should be considered as a separate criterion when developing warrants for grade-separated interchanges. Present methods of accident analysis, which relate accident experience with traffic volumes, do not provide reliable estimates of the number of accidents that would occur at proposed facilities. The relationship between vehicle and driver performance and specific design features must be better understood before it will be pos-

TABLE 5

TRAFFIC VOLUME COMBINATIONS STUDIED

Arterial Highway Average Daily Traffic	Expressway Average Daily Traffic				
	3,000	5,000	7,000	11,000	15,000
2,000	X	X	X	X	X
3,000	X	X	X	X	X
5,000		X	X	X	X
7,000			X	X	X

TABLE 6

COMPARISON OF ANNUAL TRAVEL TIMES—SIGNALIZED INTERSECTION VS DIAMOND INTERCHANGE, BASIC TRAFFIC CONDITIONS[a]

Average Daily Traffic		Total Annual Travel Time		Travel Time Savings			
Expressway (vpd)	Arterial (vpd)	Signalized Intersection (hours)	Diamond Interchange (hours)	Total Hours/ Year	Percent of At-Grade	Seconds per Vehicle	Monetary Value at $1.50/Hour
3,000	2,000	39,175	34,692	4,483	11	9	6,700
3,000	3,000	48,376	43,078	5,298	11	9	7,900
5,000	2,000	54,493	46,640	7,853	14	11	11,800
5,000	3,000	63,920	55,025	8,895	14	11	13,300
5,000	5,000	83,410	71,861	11,549	14	11	17,300
7,000	2,000	70,286	58,588	11,698	16	13	17,500
7,000	3,000	79,939	66,973	12,966	16	13	19,400
7,000	5,000	99,882	83,809	16,073	16	13	24,100
7,000	7,000	120,673	100,723	19,950	16	14	29,900
11,000	2,000	103,296	82,483	20,813	20	16	31,200
11,000	3,000	113,402	90,869	22,533	20	16	33,800
11,000	5,000	134,249	107,704	26,545	20	16	39,800
11,000	7,000	155,944	124,618	31,326	20	17	47,000
15,000	2,000	138,205	106,378	31,827	23	18	47,700
15,000	3,000	148,763	114,764	33,999	23	19	51,000
15,000	5,000	170,515	131,599	38,916	23	19	58,400
15,000	7,000	193,115	148,514	44,601	23	20	66,900

[a]Truck percentage: expressway, 28 percent; arterial, 20 percent.
Percent turns: 20 percent of arterial volume in each direction.

TABLE 7

COMPARISON OF ANNUAL VEHICLE OPERATING COSTS—SIGNALIZED INTERSECTION VS DIAMOND INTERCHANGE, BASIC TRAFFIC CONDITIONS[a]

Average Daily Traffic		Annual Vehicle Operating Costs ($/year)			
Expressway (vpd)	Arterial (vpd)	Signalized Intersection	Diamond Interchange	Annual Savings	Percent of At-Grade
3,000	2,000	208,600	149,400	59,200	28
3,000	3,000	250,800	189,500	61,300	24
5,000	2,000	293,000	195,500	97,500	33
5,000	3,000	335,700	235,600	100,100	30
5,000	5,000	421,400	315,900	105,500	25
7,000	2,000	377,300	241,600	135,700	36
7,000	3,000	420,600	281,700	138,900	33
7,000	5,000	507,600	362,000	145,600	29
7,000	7,000	594,700	442,200	152,500	26
11,000	2,000	545,900	333,800	212,100	39
11,000	3,000	590,500	374,000	216,500	37
11,000	5,000	679,900	454,200	225,700	33
11,000	7,000	769,600	534,400	235,200	30
15,000	2,000	714,400	426,100	288,300	40
15,000	3,000	760,300	466,200	294,100	39
15,000	5,000	852,200	546,400	305,800	36
15,000	7,000	944,400	626,700	317,700	34

[a]Truck percentage: expressway, 28 percent; arterial, 20 percent.
Percent turns: 20 percent of arterial volume in each direction.

sible to forecast accident rates accurately at proposed facilities. In addition, experiences at poorly designed at-grade intersections are not necessarily representative of the accident rates that would occur at a modern, properly signed channelized intersection. New methods of signing and of controlling traffic could significantly reduce the number of accidents at signalized intersections.

However, accident costs should be considered in the economic analysis. Accident data (Fig. 4) for rural signalized intersections and diamond interchanges in Illinois pro-

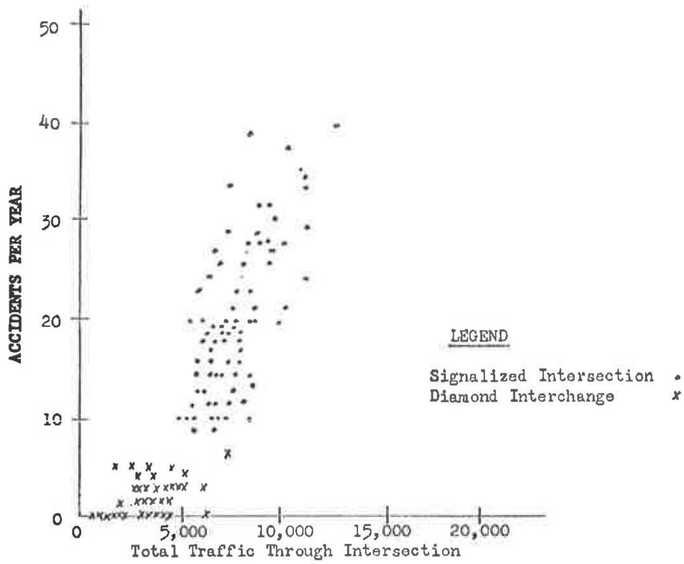

Figure 4. Comparison of accident experience in Illinois—signalized intersections vs diamond interchanges.

vided an index to the relative costs at the two intersection types. While these savings are not necessarily accurate, they do provide a rough estimate of the resulting benefits. A regression analysis was undertaken with the number of accidents as the dependent variable and the average daily traffic on the two intersecting highways as the independent variables. The savings in accident cost predicted by the regression equations for both the at-grade signalized intersection and interchange are given in Table 8.

Total User Benefits

Table 9 summarizes the user savings, expressed in monetary terms, obtained by providing a diamond interchange rather than a signalized intersection. As can be seen, savings in operating costs account for the largest percentage, ranging from 78 at the highest traffic volume combinations to 90 percent at the lowest traffic volume combinations. By assuming a value of time of $1.50 per hour, time savings accounted for less than 17 percent of the total savings at all traffic volume combinations considered while accident savings accounted for less than 7 percent of the total savings.

Comparison of Benefits With Costs

The annual benefits in user savings are compared in Table 10 with the additional cost of providing a diamond interchange for the full range of traffic volume combinations,

TABLE 8

ANNUAL SAVINGS ($) OF ACCIDENT COST RESULTING BY PROVIDING A DIAMOND INTERCHANGE RATHER THAN A SIGNALIZED INTERSECTION

Arterial Highway Average Daily Traffic	Expressway Average Daily Traffic				
	3,000	5,000	7,000	11,000	15,000
2,000	100	4,000	7,800	15,600	23,400
3,000	400	4,300	8,200	16,000	23,700
5,000		5,100	9,000	16,700	24,500
7,000			9,700	17,500	25,200

TABLE 9

SUMMARY OF ANNUAL USER SAVINGS OBTAINED BY PROVIDING A DIAMOND INTERCHANGE RATHER THAN A SIGNALIZED INTERSECTION, BASIC TRAFFIC CONDITIONS[a]

Average Daily Traffic		Annual Savings						
Expressway (vpd)	Arterial (vpd)	Operating Cost		Time Cost[b]		Accident Cost		Total ($/year)
		$/Year	Percent	$/Year	Percent	$/Year	Percent	
3,000	2,000	59,200	90	6,700	10	100	0	66,000
3,000	3,000	61,300	88	7,900	11	400	1	69,600
5,000	2,000	97,500	86	11,800	10	4,000	4	113,300
5,000	3,000	100,100	85	13,300	11	4,300	4	117,700
5,000	5,000	105,500	82	17,300	14	5,100	4	127,900
7,000	2,000	135,700	84	17,500	11	7,800	5	161,000
7,000	3,000	138,900	83	19,400	12	8,200	5	166,500
7,000	5,000	145,600	81	24,100	14	9,000	5	178,700
7,000	7,000	152,500	79	29,900	16	9,700	5	192,100
11,000	2,000	212,100	82	31,200	12	15,600	6	258,900
11,000	3,000	216,500	81	33,800	13	16,000	6	266,300
11,000	5,000	225,700	80	39,800	14	16,700	6	282,200
11,000	7,000	235,200	78	47,000	16	17,500	6	299,700
15,000	2,000	288,300	80	47,700	13	23,400	7	359,400
15,000	3,000	294,100	80	51,000	14	23,700	6	368,800
15,000	5,000	305,800	79	58,400	15	24,500	6	388,700
15,000	7,000	317,700	78	66,900	16	25,200	6	409,800

[a]Truck percentage: expressway, 28 percent; arterial, 20 percent.
Percent turns: 20 percent of arterial volume in each direction.
[b]Assumed value of time equal to $1.50 per hour.

TABLE 10

BENEFIT COST RATIOS—SIGNALIZED INTERSECTION VS DIAMOND INTERCHANGE, BASIC TRAFFIC CONDITIONS[a]

		Cost Difference											
Average Daily Traffic		$280,000				$400,000				$680,000			
Expressway	Arterial	0%		7%		0%		7%		0%		7%	
		0	$1.50	0	$1.50	0	$1.50	0	$1.50	0	$1.50	0	$1.50
3,000	2,000	3.2	3.6	1.9	2.1	2.4	2.7	1.4	1.6	1.5	1.7	0.9	1.0
3,000	3,000	3.4	3.8	2.0	2.3	2.5	2.9	1.5	1.7	1.6	1.8	0.9	1.0
5,000	2,000	5.5	6.2	3.3	3.7	4.2	4.7	2.4	2.7	2.6	3.0	1.5	1.7
5,000	3,000	5.7	6.4	3.4	3.8	4.3	4.8	2.5	2.8	2.7	3.1	1.5	1.7
5,000	5,000	6.0	7.0	3.6	4.2	4.6	5.3	2.6	3.0	2.9	3.3	1.6	1.9
7,000	2,000	7.8	8.9	4.7	5.2	5.9	6.6	3.4	3.8	3.7	4.2	2.1	2.4
7,000	3,000	8.0	9.1	4.8	5.4	6.1	6.9	3.5	4.0	3.8	4.3	2.1	2.4
7,000	5,000	8.4	9.8	5.0	5.8	6.4	7.4	3.7	4.2	4.0	4.7	2.3	2.6
7,000	7,000	8.9	10.5	5.3	6.3	6.7	7.9	3.9	4.6	4.2	5.0	2.4	2.8
11,000	2,000	12.4	14.1	7.4	8.4	9.4	10.7	5.4	6.2	5.9	6.8	3.3	3.8
11,000	3,000	12.7	19.6	7.6	8.7	9.6	11.0	5.3	6.3	6.1	7.0	3.4	3.9
11,000	5,000	13.2	15.4	7.9	9.2	10.0	11.6	5.8	6.7	6.3	7.4	3.5	4.1
11,000	7,000	13.8	16.4	8.2	9.8	10.4	12.3	6.0	7.1	6.6	7.8	3.7	4.4
15,000	2,000	17.0	19.6	10.1	11.7	12.8	14.8	7.4	8.5	8.1	9.4	4.6	5.2
15,000	3,000	17.4	20.2	10.3	12.0	13.1	15.2	7.6	8.8	8.3	9.6	4.6	5.4
15,000	5,000	18.0	21.2	10.7	12.6	13.6	16.0	7.9	9.2	8.6	10.1	4.8	5.7
15,000	7,000	18.7	22.4	11.2	13.3	14.1	16.9	8.2	9.7	9.0	10.7	5.0	6.0

[a]Truck percentage: expressway, 28 percent; arterial, 20 percent.
Percent turns: 20 percent of arterial volume in each direction.

capital costs, interest rates, and time values. Each benefit-cost ratio indicates the dollar savings that would result for each extra dollar invested in the diamond interchange. In almost all cases the benefit exceeded the extra annual cost regardless of interest rates or values of time chosen.

SENSITIVITY OF RESULTS TO TRAFFIC PARAMETERS

Additional analyses were undertaken to determine the sensitivity of the results to changes in the basic traffic parameters. Other values for the traffic parameters—

percent turning vehicles, percent trucks, and hourly traffic distribution—were input into the basic regression equations, and travel time, operating costs, and benefit-cost ratios were determined. The turning volumes in each direction were reduced from 20 to 15 percent of the arterial volume. Trucks were reduced by 5 percent. A more peaked hourly traffic variation was also tested. This distribution is compared in Table 11 with the basic hourly traffic distribution.

TABLE 11

HOURLY TRAFFIC VARIATION TESTED IN THE SENSITIVITY ANALYSIS

Hour Period	Typical Distribution		Peaked Distribution	
	a.m.	p.m.	a.m.	p.m.
12- 1	1.4	5.6	1.4	5.0
1- 2	1.2	5.9	0.8	5.4
2- 3	1.2	6.2	0.6	6.4
3- 4	1.3	6.9	0.5	8.1
4- 5	2.0	6.6	0.8	8.8
5- 6	3.4	5.8	2.5	7.4
6- 7	4.8	4.9	5.8	5.6
7- 8	5.3	4.1	6.4	4.3
8- 9	5.5	3.4	5.2	3.4
9-10	5.5	3.2	4.8	2.8
10-11	5.3	2.8	4.8	2.4
11-12	5.1	2.6	4.8	2.0

TABLE 12

COMPARISON OF TRAVEL TIME SAVINGS FOR VARYING TRAFFIC CONDITIONS—SIGNALIZED INTERSECTION VS DIAMOND INTERCHANGE

Average Daily Traffic		Annual Travel Time Savings						
		Basic Traffic Conditions[a]	Lower Percent Turning[b]		Lower Percent Trucks[c]		Peaked Hourly Distribution[d]	
Expressway	Arterial	Hours per Year	Hours per Year	Percent of Basic	Hours per Year	Percent of Basic	Hours per Year	Percent of Basic
3,000	2,000	4,483	5,107	114	4,412	98	5,036	112
3,000	3,000	5,298	6,226	118	5,252	99	6,042	114
5,000	2,000	7,853	8,480	109	7,712	98	8,717	111
5,000	3,000	8,895	9,827	110	8,770	99	9,978	112
5,000	5,000	11,549	13,088	113	11,472	99	13,156	113
7,000	2,000	11,698	12,328	106	11,497	98	12,935	111
7,000	3,000	12,966	13,902	108	12,774	99	14,453	112
7,000	5,000	16,073	17,619	110	15,913	99	18,143	113
7,000	7,000	19,950	22,100	111	19,844	99	22,714	114
11,000	2,000	20,813	21,449	103	20,523	99	22,954	110
11,000	3,000	22,533	23,447	105	22,236	99	25,413	111
11,000	5,000	26,545	28,104	106	26,299	99	29,727	112
11,000	7,000	31,326	33,495	107	31,055	99	35,322	113
15,000	2,000	31,827	32,468	102	31,488	99	35,183	110
15,000	3,000	33,999	34,952	103	33,639	99	37,724	111
15,000	5,000	38,916	40,488	104	38,529	99	43,462	111
15,000	7,000	44,601	46,789	105	44,205	99	50,081	112

[a]Truck percentage: expressway, 28 percent; arterial, 20 percent.
[b]Percent turns: 20 percent of arterial volume in each direction.
[c]Percent turns: 15 percent of arterial volume in each direction.
[d]Truck percentage: expressway, 23 percent; arterial, 15 percent.
See Table 11.

Travel Time

A comparison of savings in travel time is given in Table 12. By decreasing the percent turning by 5 percent for each movement, the savings in travel time increased between 2 and 18 percent with the largest increase occurring at the lowest traffic volume combinations. This was true because the time savings per vehicle were greater for vehicles going straight than for turning vehicles. By decreasing the turning movements, the savings therefore increased.

By lowering the percent of trucks 5 percent, the time savings decreased about 1 percent for all volume combinations studied. This was due to the fact that the time savings per passenger vehicle was less than the time savings per truck.

The time savings increased by as much as 14 percent with the more peaked hourly traffic distribution.

Operating Costs

The sensitivity of savings in operating cost to the traffic parameters is demonstrated in Table 13. The savings in operating costs responded to changes in turning movements about the same as had time savings. At the highest traffic volume combinations considered, the savings in operating cost were somewhat more pronounced than were time savings.

By lowering the percent of trucks 5 percent, the savings in operating cost decreased nearly 9 percent. The savings in operating costs realized by the average truck by building an interchange rather than a signalized intersection were much greater than they were for the average passenger vehicle.

Savings in operating costs were much less responsive to the peaked hourly traffic distribution than were the travel time savings. This was true because the operating cost per vehicle computed at the signalized intersection had a small variance and was therefore not very sensitive to changes in traffic volumes.

TABLE 13

COMPARISON OF ANNUAL OPERATING SAVINGS FOR VARYING TRAFFIC CONDITIONS—SIGNALIZED INTERSECTION VS DIAMOND INTERCHANGE

Average Daily Traffic		Annual Savings In Operating Costs						
		Basic Traffic Conditions[a]	Lower Percent Turning[b]		Lower Percent Trucks[c]		Peaked Hourly Distribution[d]	
Expressway (vpd)	Arterial (vpd)	$/Year	$/Year	Percent of Basic	$/Year	Percent of Basic	$/Year	Percent of Basic
3,000	2,000	59,200	67,700	114	54,200	92	61,400	104
3,000	3,000	61,300	74,000	121	56,100	92	63,900	104
5,000	2,000	97,500	105,900	109	89,100	91	100,500	103
5,000	3,000	100,100	112,900	113	91,600	91	103,700	103
5,000	5,000	105,500	127,100	120	96,900	92	110,400	104
7,000	2,000	135,700	144,100	106	124,000	92	140,000	103
7,000	3,000	138,900	151,700	109	127,000	92	143,500	103
7,000	5,000	145,600	167,200	115	133,400	92	151,600	104
7,000	7,000	152,500	183,100	120	140,300	92	160,000	105
11,000	2,000	212,100	220,500	104	193,600	91	217,800	103
11,000	3,000	216,500	229,300	106	197,700	91	223,100	103
11,000	5,000	225,700	247,300	110	206,300	92	234,100	104
11,000	7,000	235,200	265,700	113	215,400	92	245,200	104
15,000	2,000	288,300	296,800	103	263,000	91	296,000	103
15,000	3,000	294,100	306,900	104	268,300	91	302,700	103
15,000	5,000	305,800	327,300	107	279,100	91	316,500	104
15,000	7,000	317,700	348,300	110	290,400	92	330,500	104

[a]Truck percentage: expressway, 28 percent; arterial, 20 percent.
Percent turns: 20 percent of arterial volume in each direction.
[b]Percent turns: 15 percent of arterial volume in each direction.
[c]Truck percentage: expressway, 23 percent; arterial, 15 percent.
[d]See Table 11.

Benefit-Cost Ratios

Tables 14, 15, and 16 can be used to determine the benefit-cost ratios when the traffic parameters at an intersection under study are different from the basic traffic parameters. In all three cases the benefit-cost ratios are consistently above unity except for low traffic volume conditions having large capital cost differences.

TABLE 14

BENEFIT COST RATIOS AT LOWER PERCENT TURNING—SIGNALIZED INTERSECTION VS DIAMOND INTERCHANGE[a]

Average Daily Traffic		Cost Difference											
		$280,000				$400,000				$680,000			
Expressway	Arterial	0%		7%		0%		7%		0%		7%	
		0	$1.50	0	$1.50	0	$1.50	0	$1.50	0	$1.50	0	$1.50
3,000	2,000	3.7	4.1	2.2	2.5	2.8	3.1	1.6	1.8	1.8	2.0	1.0	1.1
3,000	3,000	4.1	4.6	2.4	2.7	3.1	3.4	1.8	2.0	1.9	2.2	1.1	1.2
5,000	2,000	6.0	6.7	3.6	4.0	4.5	5.0	2.6	2.9	2.9	3.2	1.6	1.8
5,000	3,000	6.4	7.2	3.8	4.3	4.8	5.4	2.8	3.1	3.1	3.4	1.7	1.9
5,000	5,000	7.2	8.3	4.3	4.9	5.4	6.2	3.1	3.6	3.5	4.0	1.9	2.2
7,000	2,000	8.3	9.3	4.9	5.5	6.3	7.0	3.6	4.1	4.0	4.4	2.2	2.5
7,000	3,000	8.7	9.9	5.2	5.9	6.6	7.4	3.8	4.3	4.2	4.7	2.3	2.6
7,000	5,000	9.6	11.1	5.7	6.6	7.2	8.3	4.2	4.8	4.6	5.3	2.6	3.0
7,000	7,000	10.5	12.3	6.3	7.3	7.9	9.3	4.6	5.4	5.0	5.9	2.8	3.3
11,000	2,000	12.9	14.7	7.7	8.7	9.7	11.0	5.6	6.4	6.2	7.0	3.4	3.9
11,000	3,000	13.4	15.3	8.0	9.1	10.1	11.5	5.8	6.7	6.4	7.3	3.6	4.1
11,000	5,000	14.4	16.7	8.6	10.0	10.9	12.6	6.3	7.3	6.9	8.0	3.9	4.5
11,000	7,000	15.5	18.2	9.2	10.9	11.7	13.7	6.7	7.9	7.4	8.7	4.1	4.9
15,000	2,000	17.5	20.2	10.4	12.0	13.2	15.2	7.6	8.8	8.4	9.6	4.7	5.4
15,000	3,000	18.1	20.9	10.8	12.5	13.6	15.8	7.9	9.1	8.6	10.0	4.8	5.6
15,000	5,000	19.2	22.5	11.4	13.4	14.5	17.0	8.4	9.8	9.2	10.8	5.1	6.0
15,000	7,000	20.4	24.2	12.2	14.4	15.4	18.3	8.9	10.6	9.8	11.6	5.5	6.5

[a]Truck percentage: expressway, 28 percent; arterial, 20 percent.
Percent turns: 15 percent of arterial volume in each direction.

TABLE 15

BENEFIT COST RATIOS AT LOWER PERCENT OF TRUCKS—SIGNALIZED INTERSECTION VS DIAMOND INTERCHANGE[a]

Average Daily Traffic		Cost Difference											
		$280,000				$400,000				$680,000			
Expressway	Arterial	0%		7%		0%		7%		0%		7%	
		0	$1.50	0	$1.50	0	$1.50	0	$1.50	0	$1.50	0	$1.50
3,000	2,000	3.0	3.3	1.8	2.0	2.2	2.5	1.3	1.4	1.4	1.6	0.8	0.9
3,000	3,000	3.1	3.5	1.8	2.1	2.3	2.7	1.3	1.5	1.5	1.7	0.8	0.9
5,000	2,000	5.1	5.7	3.0	3.4	3.8	4.3	2.2	2.5	2.4	2.7	1.4	1.5
5,000	3,000	5.2	6.0	3.1	3.5	3.9	4.5	2.3	2.6	2.5	2.8	1.4	1.6
5,000	5,000	5.6	6.5	3.3	3.9	4.2	4.9	2.4	2.8	2.7	3.1	1.5	1.7
7,000	2,000	7.2	8.1	4.3	4.8	5.4	6.1	3.1	3.5	3.4	3.9	1.9	2.2
7,000	3,000	7.4	8.4	4.4	5.0	5.6	6.4	3.2	3.7	3.5	4.0	2.0	2.3
7,000	5,000	7.8	9.1	4.6	5.4	5.9	6.8	3.4	4.0	3.7	4.3	2.1	2.4
7,000	7,000	8.2	9.8	4.9	5.8	6.2	7.4	3.6	4.3	3.9	4.7	2.2	2.6
11,000	2,000	11.4	13.1	6.8	7.8	8.6	9.9	5.0	5.7	5.5	6.3	3.1	3.5
11,000	3,000	11.7	13.5	7.0	8.0	8.8	10.2	5.1	5.9	5.6	6.4	3.1	3.6
11,000	5,000	12.2	14.3	7.3	8.5	9.2	10.8	5.3	6.2	5.8	6.9	3.3	3.8
11,000	7,000	12.7	15.3	7.6	9.1	9.6	11.5	5.5	6.6	6.1	7.3	3.4	4.1
15,000	2,000	15.6	18.2	9.3	10.9	11.8	13.7	6.8	7.9	7.5	8.7	4.2	4.9
15,000	3,000	16.0	18.7	9.5	11.1	12.0	14.1	6.9	8.1	7.6	8.9	4.3	5.0
15,000	5,000	16.6	19.7	9.9	11.8	12.5	14.9	7.2	8.6	7.9	8.9	4.3	5.0
15,000	7,000	17.2	20.9	10.3	12.4	13.0	15.7	7.5	9.1	8.2	10.0	4.6	5.6

[a]Truck percentage: expressway, 23 percent; arterial, 15 percent.
Percent turns: 20 percent of arterial volume in each direction.

TABLE 16

BENEFIT COST RATIOS AT PEAKED TRAFFIC DISTRIBUTION—SIGNALIZED INTERSECTION VS DIAMOND INTERCHANGE[a]

Average Daily Traffic		Cost Difference											
		$280,000				$400,000				$680,000			
Expressway	Arterial	0%		7%		0%		7%		0%		7%	
		0	$1.50	0	$1.50	0	$1.50	0	$1.50	0	$1.50	0	$1.50
3,000	2,000	3.4	3.8	2.0	2.2	2.5	2.8	1.5	1.6	1.6	1.8	0.9	1.0
3,000	3,000	3.5	4.0	2.1	2.4	2.6	3.0	1.5	1.7	1.7	1.9	0.9	1.1
5,000	2,000	5.7	6.4	3.4	3.8	4.3	4.8	2.5	2.8	2.7	3.1	1.5	1.7
5,000	3,000	5.9	6.7	3.5	4.0	4.4	5.1	2.6	2.9	2.8	3.2	1.6	1.8
5,000	5,000	6.3	7.4	3.7	4.4	4.8	5.6	2.7	3.2	3.0	3.5	1.7	2.0
7,000	2,000	8.1	9.1	4.8	5.4	6.1	6.9	3.5	4.0	3.8	4.4	2.1	2.4
7,000	3,000	8.3	9.5	4.9	5.6	6.2	7.1	3.6	4.1	4.0	4.5	2.2	2.5
7,000	5,000	8.8	10.3	5.2	6.1	6.6	7.7	3.8	4.7	4.2	4.9	2.3	2.7
7,000	7,000	9.3	11.1	5.5	6.6	7.0	8.4	4.0	4.8	4.4	5.3	2.5	3.0
11,000	2,000	12.7	14.6	7.6	8.7	9.6	11.0	5.5	6.4	6.1	7.0	3.4	3.9
11,000	3,000	13.1	15.1	7.8	9.0	9.8	11.4	5.7	6.6	6.2	7.2	3.5	4.0
11,000	5,000	13.7	16.1	8.2	9.6	10.3	12.2	6.0	7.0	6.5	7.7	3.7	4.3
11,000	7,000	14.4	17.3	8.5	10.3	10.8	13.0	6.2	7.5	6.9	8.2	3.8	4.6
15,000	2,000	17.4	20.3	10.4	12.1	13.1	15.3	7.6	8.8	8.3	9.7	4.7	5.4
15,000	3,000	17.8	21.0	10.6	12.5	13.4	15.7	7.8	9.1	8.5	10.0	4.8	5.6
15,000	5,000	18.6	22.2	11.1	13.2	14.0	16.7	8.1	9.7	8.9	10.6	5.0	5.9
15,000	7,000	19.4	23.5	11.6	14.0	14.6	17.7	8.5	10.2	9.3	11.2	5.3	6.3

[a]Truck percentage: expressway, 28 percent; arterial, 20 percent.
Percent turns: 20 percent of arterial volume in each direction.
Hourly distribution: see Table 15.

STOP-SIGN CONTROLLED INTERSECTION VS DIAMOND INTERCHANGE

At low traffic volumes, another alternative solution would be to control the traffic entering the at-grade intersection with either a two-way or four-way stop sign. Differences in operating costs and travel times between each type of stop sign control and a diamond interchange were calculated. Accident costs were not included in the economic analysis because they would constitute only a small portion of the user benefits at the traffic volumes considered. It was assumed that the stop-sign control would be replaced by either a traffic signal or a diamond interchange when the traffic volumes were high enough to warrant the installation of a signal.

Two-Way Stop Sign Control

Only vehicles approaching the intersection on the two-lane arterial highway were required to stop. All turns from the expressway were assumed to have been made at 20 mph. The basic traffic conditions of 20 and 28 percent trucks on the arterial and expressway respectively and turning movements in each direction equal to 20 percent of the arterial volume were assumed.

Travel time and user operating cost savings resulting from the construction of a diamond interchange rather than a two-way stop sign control are given in Table 17. These savings would not be significantly sensitive to changes in expressway traffic volumes at the low traffic volumes under consideration.

The user benefits resulting from the construction of a diamond interchange rather than a two-way stop are compared with the extra annual costs of building and maintaining the interchange in Table 18.

Two volume warrants for signals were investigated. The first was the "minimum vehicular warrant" established in the Manual of Uniform Traffic Control Devices (15). This warrant is satisfied when, for each of any 8 hours of an average day, the volume exceeds 420 vehicles per hour on the expressway and 105 vehicles per hour on the arterial highway. Taking into account the typical traffic distribution shown in Table 3, this warrant would be satisfied when the expressway volume is 7,000 vehicles per day and the arterial volume is 4,000 vehicles per day. At these volumes, the savings to the road user would be sufficient to offset the extra cost of constructing and maintaining a typical diamond interchange. At these volumes, the benefit-cost ratio for the inter-

TABLE 17

ANNUAL SAVINGS IN TRAVEL TIME AND OPERATING COSTS RESULTING BY PROVIDING A DIAMOND INTERCHANGE RATHER THAN A TWO-WAY STOP-SIGN CONTROLLED INTERSECTION

Arterial Highway Average Daily Traffic (vpd)	Travel Time Savings: Total (hours/year)	Travel Time Savings: Monetary Value at $1.50/Hour ($/year)	Operating Cost Savings ($/year)
500	500	750	6,600
1,000	1,000	1,500	13,100
2,000	2,000	3,000	26,300
3,000	3,000	4,500	39,400
4,000	4,000	6,000	52,500

TABLE 18

BENEFIT COST RATIOS—TWO-WAY STOP-SIGN CONTROLLED INTERSECTION VS DIAMOND INTERCHANGE

Arterial Highway Average Daily Traffic (vpd)	Cost Difference											
	$280,000				$400,000				$680,000			
	0%		7%		0%		7%		0%		7%	
	0	$1.50	0	$1.50	0	$1.50	0	$1.50	0	$1.50	0	$1.50
500	0.4	0.4	0.2	0.2	0.3	0.3	0.1	0.2	0.1	0.2	0.1	0.1
1,000	0.7	0.8	0.4	0.5	0.5	0.6	0.3	0.3	0.3	0.4	0.2	0.2
2,000	1.4	1.6	0.9	1.0	1.1	1.2	0.6	0.8	0.6	0.7	0.4	0.4
3,000	2.1	2.4	1.3	1.4	1.6	1.8	0.9	1.0	1.0	1.1	0.6	0.6
4,000	2.9	3.2	1.7	1.9	2.2	2.4	1.2	1.4	1.4	1.5	0.8	0.9

change as compared with the signal control ranged from 2.0 to 10.0, indicating that a signal-controlled intersection would not be economically justified.

The second traffic volume condition tested met the "interruption of continuous traffic warrant" (15). This warrant would be satisfied when, for each of any 8 hours of an average day, the volume on the expressway exceeds 630 vehicles per hour and the volume on the arterial exceeds 53 vehicles per hour. These volumes correspond to average daily traffic volumes of 11,000 and 2,000 vehicles per day on the expressway and arterial highway respectively. At these volumes, assuming an interest rate of 0 percent, user benefits would be more than the cost of an interchange. Again a signal could not be economically justified, where benefit-cost ratios ranging from 3.1 to 14.7 were determined for the diamond interchange vs the signalized at-grade intersection.

Four-Way Stop Sign Control

A four-way stop sign control at an at-grade intersection was also investigated as an alternative treatment. The Manual on Uniform Traffic Control Devices (15) specifies that this type of control should only be used where the volume of traffic on the intersecting roads is nearly equal. It also states that the total vehicular volume entering the intersection should be at least 350 and not more than 420 vehicles per hour for any 8 hours of an average day. These volumes would correspond to average daily traffic volumes of 3,000 vehicles per day at a minimum and 4,000 vehicles per day at a maximum for each of the intersecting highways. The operating cost is very high at a four-way stop sign control since all vehicles must stop. The operating cost alone was sufficient to indicate the undesirability of a four-way stop, with annual user savings of $105,500 at the minimum volume conditions and $140,000 at the maximum volume conditions resulting by installing a diamond interchange rather than the four-way stop sign. From an economic point of view, a four-way stop sign control would not be warranted at a rural expressway intersection having traffic volumes conforming to the warrants for four-way stop sign control.

SUMMARY AND CONCLUSIONS

The purpose of this study was to quantitatively compare the consequences to the public of constructing an at-grade intersection with the consequences of constructing a diamond interchange at an intersection between a rural expressway and an arterial highway. Two criteria, measured at various traffic volume combinations and at different traffic conditions, were selected for presentation—level of service and comparison of user benefits with construction and maintenance costs.

At-Grade Signalized Intersection vs Diamond Interchange

The running time is reduced in the range of 11 to 23 percent for the interchange as compared with the signal. This amounts to an average savings of 10 to 20 seconds per vehicle.

User benefits were taken as the sum of savings in accident costs, time costs, and operating costs saved by constructing a diamond interchange rather than using an at-grade signalized intersection. This benefit was compared with the additional annual cost of constructing and maintaining the diamond interchange. These benefit-cost ratios are presented for a wide range of traffic volumes and conditions so that for a specific condition the tables can be used to find the benefit directly. The benefit-cost ratios vary from slightly below one to almost 25. Only for a very few conditions is the benefit less than the cost.

Vehicle operating cost accounted for from 78 to 90 percent of the total user savings provided by an interchange. For almost all conditions, the operating cost alone was larger than the additional cost of providing the interchange.

Stop Sign Control vs Diamond Interchange

The benefit obtained by providing a diamond interchange rather than two-way stop sign or four-way stop sign control was calculated. When arterial volumes are low, two-way stop signs are more economical than providing an interchange. It was found that the annual benefit obtained by providing an interchange would be near or above the annual cost for the interchange when traffic volumes exceeded the minimum traffic volume warrants for a traffic signal, depending on the assumptions. For these volumes and assumptions, an interchange was economically justified rather than a signalized intersection.

For the range of traffic volumes normally warranting four-way stop control, a diamond interchange would result in much greater benefits in relation to costs than would use of four-way stop control.

Conclusions

These observations lead to the conclusion that, for the range of conditions considered, when a two-way stop sign control is no longer adequate at an intersection on a rural expressway, a diamond interchange would be warranted from the economic standpoint rather than using traffic signals or four-way stop sign control.

The benefit-cost ratios presented can be used as a guide by the decision-makers to determine whether the benefit obtained by providing an interchange on expressways rather than an at-grade signalized intersection is sufficient to warrant the additional cost. Different values of traffic volumes, interchange cost, interest rate, value of time, percent trucks, percent turns, and type of hourly traffic variation can be used in the tables. The interchanges can be ranked along with other projects by benefit-cost ratios to determine whether they should be built either singly or in sets to form a freeway.

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Discussion

CHARLES W. DALE, Highway Research Engineer, U.S. Bureau of Public Roads—This paper is one of the few to discuss a more complete list of criteria includable in establishing warrants for traffic control measures. Generally, previous lists of warrants for various traffic control devices have not emphasized the importance of the effects of the various traffic controls upon operating costs and travel time of the road user. These effects are significant.

The authors state that the unit operating costs used in the analysis assembled by Winfrey included a 10-cent fuel tax. This is not true. Winfrey did not include fuel taxes in his 1963 publication of vehicle operating cost tables.

Fuel taxes are a part of the out-of-pocket costs of operating a passenger car and may affect the user's decision to drive or not. However, fuel taxes are also the source of the funds for the construction and maintenance of highways. Therefore, in economy studies where the investment cost of the highway improvement is being compared with the effects on the road user, to include fuel taxes in the running cost of the vehicle would result in double counting, i.e., taxes would be included both in the highway costs and the road user costs.

In the calculation of the benefit-cost ratios, the authors use the formulation as presented in the AASHO Redbook (18), i.e., including the maintenance cost of the highway in the denominator, thus treating maintenance as a highway cost. This procedure, although not incorrect, is not preferred by many writers in the field today. Current thinking calls for including the maintenance cost for the highway in the numerator as a deduction from gross road user benefits, thus comparing all annual consequences of the improvement with the investment cost of that improvement.

Although the title of the paper is restrictive in nature (a discussion of the criteria to be used in developing warrants for rural interchanges) the authors amplified their discussion to include the analysis of an "average" intersection with varying ADT's and traffic controls, and included a sensitivity analysis using various interest rates. The mechanics of the analysis point up the inherent absence of the right types of traffic performance data needed for analyses of the economy of highway improvement proposals. In almost any and every economy analysis that one undertakes in highway and traffic engineering, the analyst comes face-to-face with the ubiquitous lack of traffic operational data. [This is discussed in greater detail elsewhere (19).]

One reservation is in order concerning the application of the results obtained in this analysis to a specific intersection or interchange analysis. Contrary to what the authors state in their paper, the decision-maker should not use the benefit-cost ratios listed in Tables 14, 15, 16, and 18 as "answers" for a study at a specific location. Each intersection, to a certain extent, is unique and traffic operational data will not, in all probability, match (e.g., different distribution and composition of traffic and different delay times) those used in this report. Then, too, there are the variations between the specific location and the assumptions used in the Gerlough-Wagner simulation program that would have a profound effect on the road user costs.

Consideration must also be taken of the judgments exercised by the authors in the tabulation of traffic accident costs and highway maintenance costs and the controversy surrounding the use of any specific rate of interest, value of travel time, and length of analysis period. Therefore, the primary values of this paper to the analyst (as I see it) are detailed discussions of the method of analysis and a listing of the inputs that are necessary for a complete analysis for economy of intersection design and not that of a reference for a quick, "cookbook" answer to specific problems.

These points I have mentioned are not meant to detract from the value of the paper. The differences are relatively insignificant when compared with the value to be derived if in the determination of warrants for traffic control devices more cognizance is given to the effects of the various traffic controls on the road user's operating costs and travel time.

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JOSEPH C. CORRADINO, Simpson and Curtin, Philadelphia—I would like to devote my discussion to the time and accident cost criteria employed by the authors in developing warrants for interchanges, and the sensitivity of their model to these criteria.

Time Cost

I agree with the authors that the value to be given time in an analysis such as they have undertaken is a controversial issue on which too little research has been performed. However, I feel the following technique, if researched, can aid in determining time costs that are more sensitive to traffic conditions than those ($0.00-$1.50) used by the authors in their analysis.

The value of time is a function of trip purpose. For non-business purposes—social-recreational, shopping, and so on—time cost may be ignored, while for business purposes, the cost of time should be related to the occupation of the vehicle operator. The determination of the value of time becomes contingent, then, upon the determination of the economic worth of the tripmaker for trips of a business nature. Combining this relationship with an arrival pattern of trips by purpose at an intersection or interchange will yield a value for time which would fit into the authors' technique.

Accident Cost

Accident cost is a function of the type and severity of an accident; however, it is also highly sensitive to the time and place at which the accident occurs. Table 19 compares the authors' accident and accident-cost figures for Illinois with similar data from studies in Massachusetts and Utah. The data for Massachusetts are dated 1953; Utah, 1955; and Illinois, 1958.

As can be seen, there is a remarkable consistency in the distribution of accidents when classified according to severity, regardless of the time or place of occurrence. However, the accident costs differ considerably from place to place and period to period. The differences are attributed to variations in factors such as population density, travel speeds, urban characteristics, and rising costs with the time and location of accidents. The authors' technique is insensitive to variability in these factors and it is felt that modifications should be made to make it reflect accident conditions unique to location and time.

Sensitivity

The final issue I wish to discuss is the sensitivity of benefit-cost ratios to savings in operating, accident, and time costs realized in constructing an interchange or an intersection where expressway and arterial meet.

Figures 5 and 6 illustrate the changes in benefit-cost ratios when a $400,000 difference between interchange and intersection construction cost is compared with savings in (a) operating costs; (b) operating costs plus time costs; and (c) operating costs plus time costs plus accident costs. Comparisons are made at interest rates of both 0 percent (Fig. 5) and 7 percent (Fig. 6).

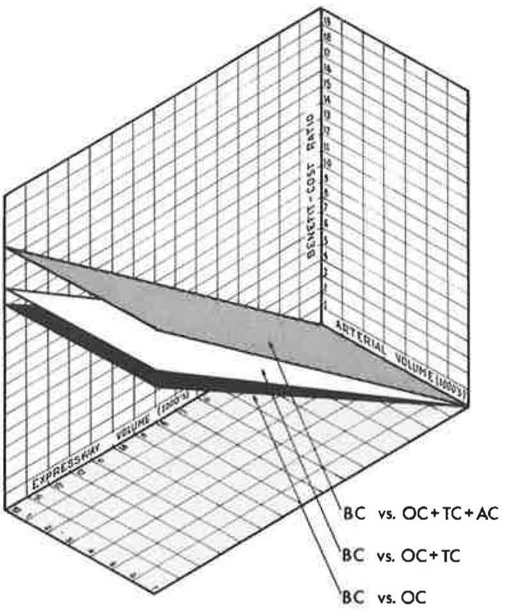
As can be seen, the three surfaces generated are almost one at low expressway volumes (less than 5,000 vehicles per day), because benefit-cost ratios differ by less than 20 percent whether all three savings are considered or only those in operating costs. As volumes increase, the surfaces diverge as the benefit-cost ratios become more sensitive to savings in time and accident costs. However, all benefit-cost ratios computed at high expressway volumes (larger than 5,000 ADT) so emphatically favor interchange construction that the increasing sensitivity of benefit-cost ratios to time and accident cost savings is without much influence. As a result it would appear that, al-

TABLE 19

COMPARISON OF ACCIDENT COST DATA*

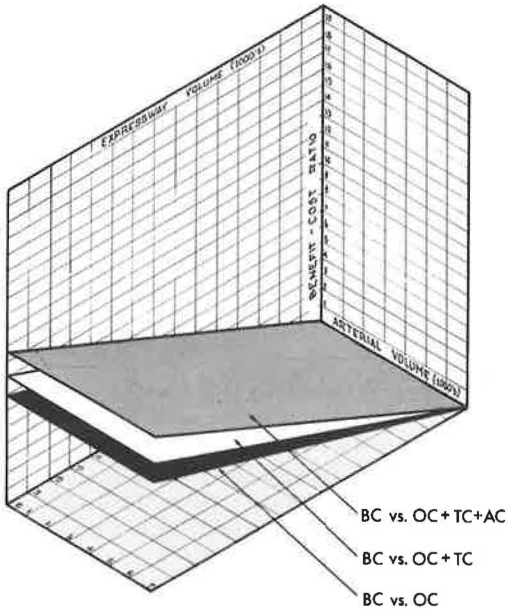
Accident Severity	Number of Accidents			Percent of Total			Direct Cost Per Accident ($)		
	Mass.	Utah	Ill.	Mass.	Utah	Ill.	Mass.	Utah	Ill.
Fatal-injury	315	77	13	0.2	0.2	0.2	5,213	3,690	7,272
Non-fatal-injury	33,270	9,048	1,958	25.3	19.0	26.8	862	1,277	1,780
Property-damage-only	97,951	38,453	5,322	74.5	80.8	73.0	203	299	186
All accidents	131,536	47,579	7,293	100.0	100.0	100.0	382	491	626

*Massachusetts and Utah data from Ref. 20, Illinois data from Ref. 8.



OC = OPERATING COST SAVINGS
TC = TIME COST SAVINGS
AC = ACCIDENT COST SAVINGS

Figure 5. Sensitivity of benefit-cost ratios to road-user savings at 0 percent interest rate.



OC = OPERATING COST SAVINGS
TC = TIME COST SAVINGS
AC = ACCIDENT COST SAVINGS

Figure 6. Sensitivity of benefit-cost ratios to road-user savings at 7 percent interest rate.

though the authors' model closely simulates real-world conditions by considering savings in operating, time, and accident costs, virtually the same results can be achieved through the use of vehicle operating costs alone.

It is also significant to note that the benefit-cost ratios developed led the authors to conclude that it is almost never economically feasible to construct a signalized intersection where expressway and arterial meet, for the conditions considered. Even at lowest volumes of 2,000 vehicles per day on the arterial and 3,000 vehicles per day on the expressway, the interchange appears to be economically better. This seems to be an impractical conclusion which may stem, in part, from the traffic signal timing and costs employed in this analysis. Perhaps the use of pre-timed rather than traffic-actuated signals has caused delays which adversely affect vehicle operating costs at intersections. Although a pre-timed signal set according to Webster's optimum timing formula may cause stopped-time and total delay to closely approximate that found at intersections controlled by traffic-actuated signals, the effects on vehicle operating costs of even small differences in stopped-time and delay, accumulated over thousands of vehicles per year, can cause quite a significant deviation from the optimum.

Also, the $120,000 cost of signals at a single intersection seems much too high. In comparison, fully interconnected, multi-phase, volume-density signals were installed at three locations in suburban Philadelphia at a total cost of $45,000. It would seem, therefore, that a reevaluation of signal cost and, perhaps, timing, would be appropriate in order to insure the practicality of the conclusions the authors have developed.

In conclusion, it should be noted that these remarks are intended as constructive criticism in hope of stimulating further research in developing warrants for interchanges on rural expressways.

Reference

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Evaluation of the Operational Effects of an "On-Freeway" Control System

JOSEPH A. WATTLEWORTH and CHARLES E. WALLACE, Texas Transportation Institute, Texas A&M University

ABRIDGMENT

•THIS report contains the results of several studies and analyses made for the purpose of evaluating the effectiveness of the "on-freeway" traffic control system on Detroit's John C. Lodge Freeway. This traffic control system consists of a group of overhead lane-use control signals and variable-message speed-control signs on a 3.2-mile section of urban freeway. To achieve this evaluation, an off-peak period set of studies, a peak-period set of studies, and a traffic system analysis were made.

Since the studies were made on only one traffic control system on one particular freeway, the results should not be viewed as necessarily applying to the general concept of "on-freeway" controls. It was found that motorists do not decrease their speeds to coincide with the posted speed unless there is an apparent reason to do so. The variable-speed signs were not successful in increasing the flow rate at a critical bottleneck. The effectiveness of the overhead lane-control signals appears to be a function of the freeway demand.

Paper sponsored by Committee on Freeway Operations and presented at the 47th Annual Meeting.

Development and Evaluation of a Ramp Metering System on the Lodge Freeway

JOSEPH A. WATTLEWORTH, CHARLES E. WALLACE, and MOSHE LEVIN, Texas Transportation Institute, Texas A&M University

ABRIDGMENT

•AN evaluation of the effectiveness of the ramp control system on Detroit's Lodge Freeway indicated that traffic operation on the freeway in the peak period was substantially improved when the control system was implemented. Statistics indicate that both total input to the freeway and total travel were not changed by the controls.

Total travel time on the freeway was about 23 percent less during the control period, indicating greater efficiency. This efficiency represents a dollar saving to freeway motorists of about $382,500 per year.

When a freeway ramp control plan is being developed, it is necessary to consider the freeway and the corridor area as an integral system. This same traffic network should also be considered in the evaluation of a control system.

Paper sponsored by Committee on Freeway Operations and presented at the 47th Annual Meeting.