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51	Highway Safety
53	Traffic Control and Operations

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CONTENTS

FOREWORD

The papers in this RECORD deal with varied aspects of freeway operations and control. They contain findings of practical value to traffic engineers and others responsible for operating freeways. Some of the material will also be of interest to highway designers, transit authorities, transportation administrators, and people involved in the performance of hazardous maintenance operations in freeway roadways.

DeCabooter and Sinha used computer simulation to compare the driver's visual task during merging maneuvers from right and left entrance ramps. Rather predictable advantages were confirmed for right ramps. They report that the model can be used to test alternative ramp designs and to analyze any on-ramp for visual quality.

In an extension of work reported earlier, Wright and Arrillaga simulated cross-median crashes at angles of 10 and 50 deg; their earlier work had been limited to right-angle crossings. They found that the probability of crash and the average impact speed are strongly related to crossing angle, median width, and speed of the crossing vehicle. Not so strongly related were other variables such as lane volume, speed of opposing vehicles, perception-reaction time, and skid resistance.

Congestion and ramp queues in a Houston interchange between 2 freeways led Loutzenheiser and Henderson to experiment with an on-freeway lane closure for brief periods as an operational measure. They conclude that the technique appears to be an effective way of reducing interchange congestion, although total delay might not necessarily be improved.

Capelle, Wagner, Hensing, and Morin report on their feasibility and evaluation study of reserved freeway lanes for buses and car pools. It was desired to determine whether more people could be moved in fewer vehicles and operations on a given freeway improved and to develop a plan for a demonstration of the effectiveness of the reserved-lane concept. They conclude that the concept is sound and warrants further study through field demonstration.

Research into safety and traffic operations attendant to various freeway lane-drop configurations was undertaken by Goodwin and Lawrence through aerial photography. Their report allowed some distinctions to be made about relative safety and efficiency and will be of interest to all who are considering the same study techniques for freeway operations.

Pretty reports on work in the Lodge Freeway corridor in Detroit relative to a ramp-metering strategy coupled with variable-message signs. The signs can be used to divert traffic to alternate routes, as well as to the freeway, depending on conditions at the freeway entrance ramp. Two discussions and the author's response help to extend the reader's understanding of the study findings.

A means of determining the average speed of groups of vehicles passing a single presence detector was investigated by Mikhalkin, Payne, and Isaksen. They report success in developing estimators capable of determining the average speed of some 20 vehicles to within 4 mph.

In the final paper, Brewer reports on experiments using forced weaving through cones and barricades as a means of controlling speeds through maintenance areas on the Interstate System. The cone and barricade pattern was determined to be highly effective when construction activity was also evident to the drivers and did not introduce unusual hazard to those traversing the area.

COMPARISON STUDY BY COMPUTER SIMULATION OF THE DRIVER'S VISUAL PART-TASK DURING LEFT AND RIGHT FREEWAY MERGING MANEUVERS

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The purpose of this study was to determine whether there is a quantifiable difference in the mechanical elements of the driver's visual part-tasks associated with left on-ramp merges as compared with mirror-image right on-ramp merges. The visual part-task is hindered by the physical limitations of gross movements of the head due to the muscular structure of the human body. Gross horizontal eye movement, or the angular movement of the visual centerline of regard, is consequently constrained. Horizontal and vertical vision is further restricted because of freeway and ramp geometry and obstructions resulting from the physical dimensions of vehicle. A general computer model was developed to simulate dynamically the visual part-task associated with the merging maneuvers. For both left and right merging, there were 6 geometric configurations considered. The results as obtained from the simulation runs clearly showed that there is a significant difference in the ramp driver's ability to see the vehicles traveling on the freeway when he is merging from the left and when he is merging from the right. In addition, the closer a driver is to the ramp nose in the dilemma zone before he is allowed to see the freeway, the less will be the chance that he can see the critical freeway vehicle before he merges. The model can be used not only to test alternative ramp designs but also to analyze individual on-ramps for visual quality.

•THE NORMAL design convention in this country is to have ramp traffic merge with freeway traffic from the right. Sometimes, however, highway engineers are forced to design left entrances because of right-of-way restrictions and the desire for interchange compactness in horizontal and vertical interchanges. Whatever the reasons, safety must be a consideration when the various determinants of interchange design are weighed. The driver's vision and ability to merge safely may be adversely affected by this type of freeway geometry.

This study describes the development and results of a computer simulation model that examines and compares quality of vision from automobiles that are traveling on left ramps and right ramps to determine whether left on-ramps do, in fact, present problems to drivers while merging.

BASIC CONSIDERATIONS

Mechanical functions associated with visual dynamics are referred to as the driver's visual part-task. The simulation model developed in the present study investigates the visual part-task of a ramp driver as he attempts to merge into the adjacent freeway lane. The part-task is at the core of the entire process of control of an automobile; indeed its activation forms the basis for all subsequent decisions that are made instinc-

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tively or deliberately to control the automobile and avoid vehicular physical contact. A ramp driver perceives the freeway automobile traffic movement through the visual stimulus of the independent movement of a particular freeway vehicle. That is a complex phenomenon in that it not only includes the relative movement of the freeway vehicles with respect to the driver himself but also involves a basic assessment of the velocities of those freeway vehicles with respect to the lane in which they are moving. However, because a driver must divide his visual attention among a number of driving tasks while he is merging, he relies heavily on his ability to estimate the relative velocities of the vehicles ahead of and behind which he will merge by turning his head and glancing at the freeway vehicles rather than by fixing his gaze on them.

Michaels (1) has observed that the detection of relative velocity depends on the rate of change of angular motion of an image across the retina of the eye. Therefore, there is a threshold for awareness of relative velocity. That threshold value was found to be 6×10^{-4} rad/sec. Only when the rate of change has reached this magnitude is relative velocity perceived. When the angular rate of change is less than this value, as is the case when a ramp driver actually accomplishes his merge, angular velocity cannot be detected. Because of that condition it was assumed that a driver will begin glancing at the freeway as soon as he feels he can see elements of the freeway vehicles so that he can begin setting up his merge maneuver at the earliest possible opportunity.

As a driver proceeds along an entrance ramp to a freeway, he ceases mentally to drive on only one roadway. Physically he is driving on the ramp roadway all the while he is upstream from the ramp nose. Downstream from that point the ramp roadway and the adjacent freeway lane become a single, variable-width lane until the end of the ramp taper is reached. The merging maneuver is assumed to be completed at or upstream from the termination of the taper.

As the driver completes his merge alignment maneuver, the side projection value (2) of the vehicles immediately ahead and behind approaches zero. Therefore, once the ramp driver decides to accept a gap between the first vehicle behind and the first vehicle ahead on the freeway lane, he becomes, for all practical purposes, a car in the freeway stream. Consequently, it was assumed that the ramp nose represents the psychological termination of the decision process and the commitment to action. Therefore, in the present study, the ramp nose was referred to as the termination of the high-speed merge dilemma zone. The length of the dilemma zone was assumed to be 300 ft. The justification of this assumption and the discussion of detailed derivations of the model operation are presented in another publication (8).

COMPONENTS OF THE MODEL

The proposed merge-vision simulation model integrates 4 groups of parametric data in an effort to quantify the visual kinesthetic responses to external stimuli required of a ramp driver as he attempts to merge with the freeway traffic stream moving in the adjacent freeway lane. The 4 groups of data that were used as input to the model are freeway and ramp geometric configuration, ramp and freeway vehicle characteristics, driver's physical measurements, and traffic characteristics.

Freeway and Ramp Geometric Configuration

The horizontal and vertical elements of the merging zones that were considered in this study are shown schematically in Figure 1. Although any quantifiable ramp terminal configuration could be handled, it was decided to include only the following 3 types of system geometrics in this study because they are the most common types in use. They are show in Figure 2.

1. Opposing sense freeway and ramp curvature—This ramp terminal could be characterized by 2 circles that meet tangentially at only 1 point and whose centers of curvature lie outside one another. The traffic on the ramp approaches the tangent point on the curvature that is of the opposite sense of that approaching the freeway. For the purpose of this study, a 3-deg curve was chosen for the ramp and a 2-deg curve was chosen for the freeway. This approximates the extreme condition allowed for a ramp entrance terminal under AASHO specifications (3).

Figure 1. Horizontal and vertical elements of merging zones.

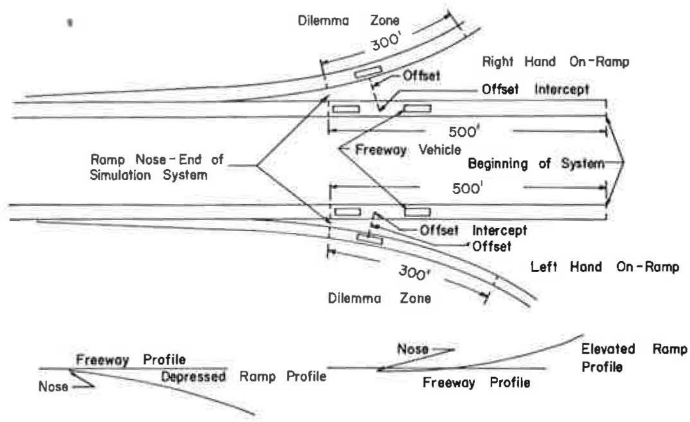

Figure 2. Types of system geometrics considered for the model.

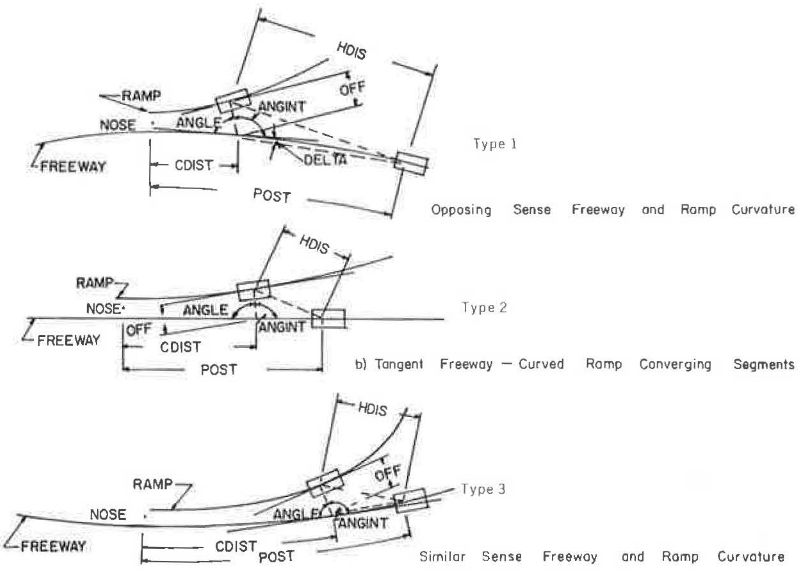

Table 1. Freeway-ramp geometric input data.

Geometric Type and Merge	Ramp Control Point	Offset Distance (ft)	Distance Upstream From Ramp Nose to Offset Intercept Point on Freeway (ft)	Acute Intersection Angle of Offset Line (deg)	Relative Pavement Elevation Difference (ft)	Geometric Type and Merge	Ramp Control Point	Offset Distance (ft)	Distance Upstream From Ramp Nose to Offset Intercept Point on Freeway (ft)	Acute Intersection Angle of Offset Line (deg)	Relative Pavement Elevation Difference (ft)
1, above	1	143.52	331.80	63.083	3.26	1, below	1	143.52	331.80	63.083	2.07
	2	118.01	272.55	65.895	1.63		2	118.01	272.55	65.89	1.92
	3	95.89	215.27	68.673	0.33		3	95.89	215.27	68.673	1.80
	4	76.90	159.64	70.403	-0.64		4	76.90	159.64	70.403	1.71
	5	60.82	105.37	74.117	-1.27		5	60.82	105.37	74.117	1.64
	Nose	47.47	52.234	76.800	-1.57		Nose	47.47	52.23	76.800	1.61
2, above	1	102.04	317.71	73.271	3.26	2, below	1	102.04	317.71	73.271	2.07
	2	87.02	263.10	74.792	1.63		2	87.02	263.10	74.792	1.92
	3	73.58	209.26	76.312	0.33		3	73.58	209.26	76.312	1.80
	4	61.70	156.12	77.833	-0.64		4	61.70	156.12	79.355	1.71
	5	51.31	103.59	79.355	-1.27		5	51.31	103.59	79.355	1.64
	Nose	42.38	51.57	80.875	-1.57		Nose	42.38	51.57	80.875	1.61
3, above	1	60.35	308.37	83.655	3.26	3, below	1	60.35	308.37	83.655	2.07
	2	54.82	256.56	84.110	1.63		2	54.82	256.56	84.110	1.92
	3	49.73	204.94	84.577	0.33		3	49.73	204.94	84.577	1.80
	4	45.08	153.49	85.049	-0.64		4	45.08	153.49	85.049	1.71
	5	40.86	102.20	85.522	-1.27		5	45.86	102.20	85.572	1.64
	Nose	37.08	51.04	85.920	-1.57		Nose	37.08	51.04	85.920	1.61

2. Tangent freeway-curved ramp converging segments—In this geometric type, the curved ramp meets tangentially with the straight line representing the alignment of the freeway. Ramp curvature remains the same as in the previous case.

3. Similar sense freeway and ramp curvature—This terminal condition is exemplified by 2 circles that meet tangentially at only 1 point and whose centers of curvature lie inside one another. The traffic on the ramp approaches the tangent point on the curvature that is of the same sense as that of the freeway. The magnitude of the arc defining degrees of curvature for both roadways was taken to be the same as that given above for opposing sense.

The geometric input data for the model were generated by use of the COGO program (4) loaded on an IBM 360/MP65 digital computer. Those data consist of distances between the assigned locus of points of the driver's vision and the path of the leading edge of the freeway vehicle. In addition, the information includes the angle of intersection of the offset line with the tangent to the freeway vehicle's path at the point of intersection. The offset line is measured at right angles to the ramp vehicle's trajectory at predesignated control points along the ramp. The digital description that was prepared for geometric input to the model is given in Table 1. The input data may be applied to both left and right on-ramps with similar entrance terminal design characteristics.

Vehicle Characteristics

Several parameters involving the vehicles on the freeway and the ramp need to be considered in the formulation of the model. It was assumed that vehicle parameters are normally distributed with a given mean and variance. Accordingly, the required vehicular characteristics were randomly assigned to individual vehicles through a Gaussian random number generator. The characteristics that were considered are discussed in the following paragraphs.

Freeway Vehicle Parameters—Only 2 structural parameters of the freeway vehicles were included: the freeway vehicular clearance over the highway pavement and the overall vehicular height. When a ramp driver approaches the freeway from below, the leading edge of the freeway vehicle first visible would be the front bumper. The bumper height can be taken as the vehicle clearance. Similarly, the vehicle's overall height defines the distance from the freeway pavement to the top edge of the windshield cowling, or the leading edge of the freeway vehicle first visible to the ramp driver approaching from above.

Ramp Vehicle Parameters—To examine microscopically the driver within the automobile capsule, one must adequately describe the cockpit as it relates to the driver. Those elements that will have a direct effect either on the driver's position or on his field of vision were determined, and the necessary data pertaining to those elements were collected. Those parameters that were judged to have an influence on the driver's position or vision within the automobile (5) are shown schematically in Figure 3. The values of the vehicle parameters are given in Table 2.

Driver's Physical Measurements

Two sets of physical measurements of major significance to the visual part-task were considered in this study. They are the driver's eye height and the gross horizontal angle through which a driver may turn his visual centerline of regard. Because most human factors are normally distributed, these 2 parameters were also assumed to follow a normal distribution. Accordingly, an attempt was made to obtain the values for the means and variances of the 2 parameters; the values are given in Table 3.

Driver's Eye Height—The dimensions of the segments of the Alderson 50th percentile male anthropometric dummy (6) were used as the basis for the measurement of the driver's eye height (EYEHT). As shown in Figure 4, vertical plane, this dimension was found to be 34.89 in. measured from the H-point to what has been considered as the geometric center of vision within the driver's head. An effort was made to secure other percentile measurements as well as information for male and female drivers

Figure 3. Vehicle input parameters.

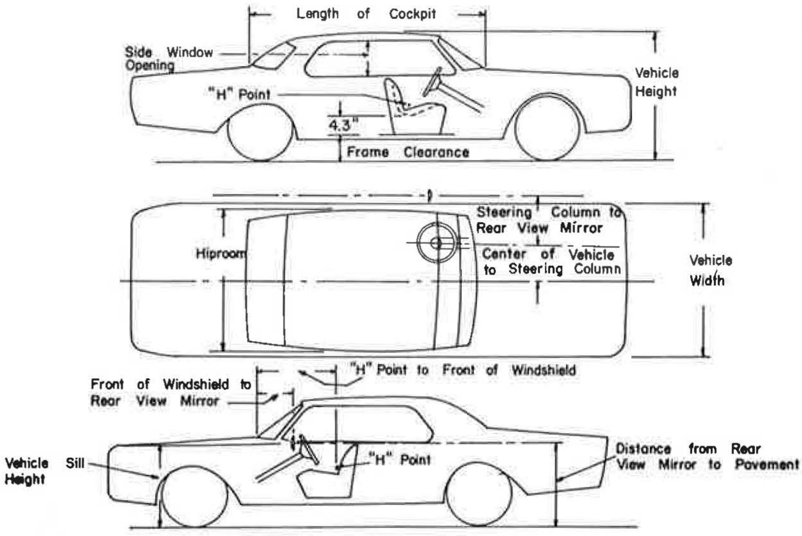

Figure 4. Orientation of driver in vehicle.

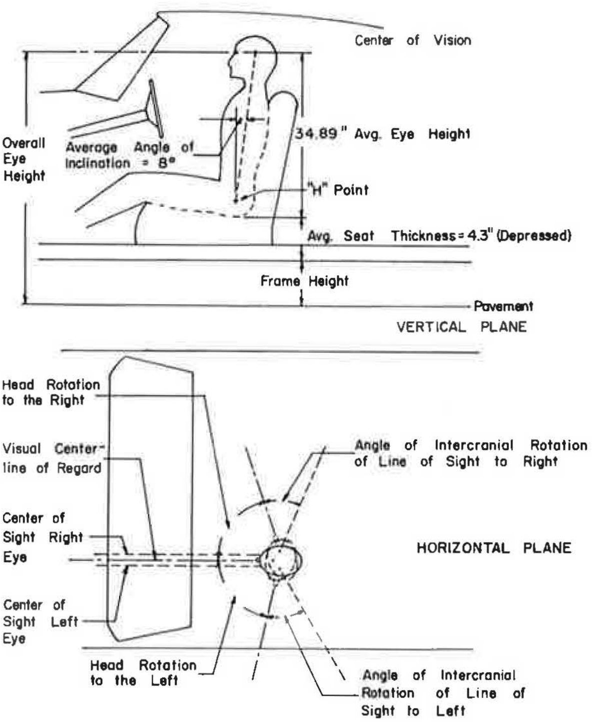

Table 2. Parameters relating to vehicle characteristics.

Parameter Dimension (in.)	Mean (in.)	Standard Deviation (in.)
Vehicle sill height[a]	37.6	1.1
Side window opening[a]	11.8	0.8
Frame to ground clearance[a]	6.3	0.4
Length of cockpit[a]	102.7	6.6
H-point to front of windshield[a]	30.9	1.7
Distance from steering centerline to vehicle centerline[a]	14.4	0.7
Vehicle hip room[a]	58.8	1.5
Vehicle overall width[a]	71.8	3.8
Vehicle overall height[a]	54.3	1.5
Distance from face of outside rearview mirror to front of windshield[b]	18.6	4.9
Distance from centerline of steering column and centerline of regard of rearview mirror[b]	19.5	2.0
Distance from pavement to centerline of regard of rearview mirror[b]	39.2	1.3

[a]Based on sample of 12 vehicle models (1969 AMA specifications).
[b]Based on field measurements of 20 vehicles ranging in age from 1 to 9 years.

Table 3. Parameters relating to driver physiology.

Parameter	Mean	Standard Deviation
Eye height[a], in.	34.9	3.5
Head rotation angle[b], deg		
Left	67.8	7.96
Right	63.9	7.77

[a]Based on 50th percentile of anthropometric dummy.
[b]Based on 40 male drivers, 120 measurements.

separately. However, because of lack of readily available information, it was assumed that the driver's eye-height measurements would fall within a range of ±20 percent of the 50th percentile value, with a 95 percent level of confidence. The standard deviation of the driver's eye-height measurement was, therefore, taken to be 3.56 in.

Gross Horizontal Visual Scan to Left and Right—The angular components of horizontal vision are shown in Figure 4, horizontal plane. To obtain the values of the parameters relative to the gross horizontal visual scan required that tests be conducted on a group of representative drivers to obtain an adequate range of angles through which these drivers could rotate their heads in the horizontal plane both to the right (DHROTR) and to the left (DHROTL). Forty randomly selected male drivers ranging in age from 22 through 53 years were tested in a Cervigon apparatus designed to measure this angle. This apparatus, manufactured by the Kitt Company of Raleigh, N. C., consisted of a large protractor etched on a shoulder rest on which was mounted a rotating chin rest and pointer.

Traffic Characteristics

The traffic flow on the freeway lane adjacent to the ramp was simulated by a shifted exponential distribution of intervehicular headways. The initial preloading of the freeway lane was accomplished by the use of a Poisson distribution of distance spacings. The speeds for the vehicles on the freeway lane were randomly assigned on the basis of a normally distributed speed model. The parameters of the freeway lane speed model were developed on the basis of the information given in the 1965 Highway Capacity Manual (7). The ramp vehicles were also assigned with randomly generated operating speeds according to a normal distribution.

MODEL OPERATION

The simulation logic was divided into 2 parts to simplify the programming of the visual kinematics. Separate programs were prepared for right and left merge situations. The simulator follows an event scan procedure; the system is evaluated only when the ramp vehicle reaches one of the predetermined control points along the ramp dilemma zone.

As a ramp vehicle is introduced into the system at the upstream end of the ramp, relative positions of the ramp vehicle and the closest freeway vehicle behind it are computed, and control is passed to the subroutines containing the logic to test the ramp driver's physical ability to perceive the freeway vehicle. All possible modes of vertical and horizontal vision are checked at each ramp station to determine whether the ramp driver can or cannot see the freeway vehicle from his vehicle while he is traveling in a given terminal geometry. When all prescribed tests have been conducted on the ramp driver, the ramp vehicle is moved forward to the next control point. After the specified number of ramp vehicles are processed through all the control points up to the ramp nose, a printout is obtained of the stored characteristics that describe the visual effects that the type of geometry being investigated has on ramp drivers attempting to make the merge.

The procedure is repeated for each of the 12 ramp geometric configurations (6 left and 6 mirror-image right ramps). The logic associated with the examination of the visual quality related to the 4 major types of ramps is discussed in the following paragraphs.

Left On-Ramp to Freeway From Below

For this merge, right-side and rearward visual scans are of the greatest importance. For a left merge from below, vision in the vertical plane may be precluded by the top of the vehicle door. In order for the ramp driver actually to be able to see the freeway vehicle in the vertical plane, he must be able to see its leading edge beneath the top of the vehicle door. In other words, the vertical angle between the driver's horizon and the leading edge of the freeway vehicle must be less than or equal to the vertical angle between the driver's horizon and the top of the door frame. The situations are shown in Figure 5. Similarly, the ramp driver's horizontal view of the freeway vehicle may be impaired by the vehicle framework if the angle required for the driver to turn his head to see the freeway vehicle is greater than the sideward view allowed by the vehicle's horizontal window opening. As an added condition for perception of the freeway vehicle, the driver must be physically able to turn his head through an angle greater than or equal to the angle between his vision line ahead and the line between him and the freeway vehicle. Vertical and horizontal visual angles are computed and compared to the angles made by the driver's sight line and the potential obstructions to determine whether the ramp driver can see the freeway vehicle from his position on the ramp.

Left On-Ramp to Freeway From Above

This merge situation is the same as the preceding situation except that, in this case, vertical vision downward to the freeway vehicle may be obstructed by the vehicle doorsill. If the vertical angle downward is obstructed, the driver is assumed not to be able to see the freeway vehicle in the vertical plane. All other aspects of this situation are similar to those of the previous case. The angular computations and comparisons are made in the same manner as defined before.

Right On-Ramp to Freeway From Below

This is the reverse or mirror-image situation of the left ramp to the freeway from below. In right merge logic, it is assumed that no horizontal obstructions to left vision are caused by the vehicular structure because of the close proximity of the driver to the left side windows. The only limitations to horizontal vision would be the driver's physical ability to rotate his head and eyes to the left. However, the top of the driver's side door may act as an obstruction to vertical vision. If the ramp driver is to be able to see the freeway vehicle in the vertical plane beneath the top of his door, the allowable upward vertical vision angle must be less than or equal to the actual vertical angle between the driver's horizon and the freeway vehicle.

Right On-Ramp to Freeway From Above

This is the reverse of the left ramp approaching the freeway from above. However, it is the same as the previous case except for 2 important differences:

1. Side vision in the vertical and horizontal planes should be almost unconstrained. Therefore, only the driver's physical ability affects visual perception of the freeway vehicle. The vehicle framework should offer no obstructions to vision.
2. Rear vision is possible. Accordingly, the horizontal and the vertical scans of the outside left rearview mirror are computed only for this type of situation.

The procedure to determine whether the freeway vehicle falls within the horizontal rearward vision cone of the ramp vehicle's rearview mirror involves comparing the angle between the ramp and the freeway vehicle to the horizontal angle between the centerline of the rearview mirror and the driver's sight center. Figure 6 shows that, if the rear view horizontal angle (the angle between the ramp and the freeway vehicle) is greater than 90 deg, the ramp driver can see the freeway vehicle in his rearview mirror in the horizontal plane. Similarly, if the vertical angle over the vehicle doorsill is less than or equal to the rear view vertical angle, the ramp driver can see the freeway vehicle in his rearview mirror in the vertical plane.

MODEL RESULTS

For left or right merging, there were 3 types of system geometrics considered for both the above and the below situations. The results as obtained from the simulation runs are discussed below.

Merge Vision

Left On-Ramp to Freeway From Above—Table 4 gives the number of drivers that can successfully see the freeway vehicle of concern in horizontal and vertical planes as well as in both planes under each freeway-ramp geometric configuration. More

Figure 5. Elements of driver's vertical sideward vision from vehicle.

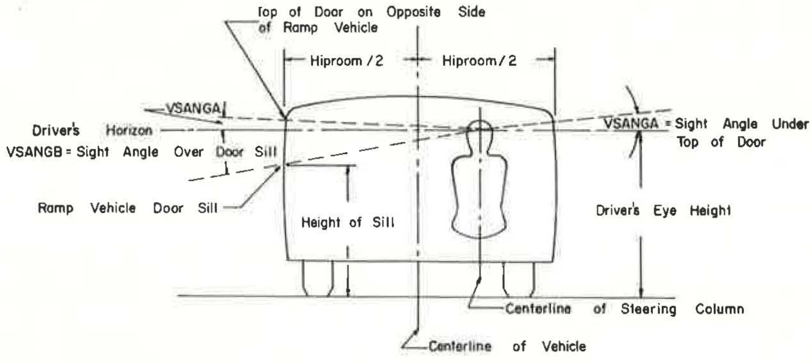

Figure 6. Geometric elements of rear vision.

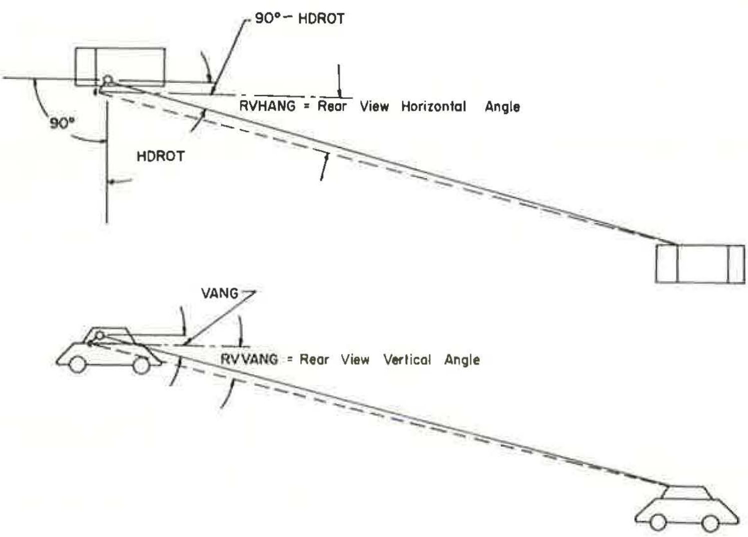

Table 4. Drivers who can successfully see freeway vehicle while they merge from above left.

Geometric Type	Ramp Control Point	Vertical Plane Vision	Horizontal Plane Vision	Clear Vision (both planes)	Number of Opportunities	Drivers Able to See Clearly (percent)
1	1	378	308	288	401	71.8
	2	436	389	274	457	59.9
	3	460	260	250	479	52.1
	4	471	234	223	489	45.6
	5	477	198	187	494	37.8
	Nose	478	131	127	496	25.6
2	1	393	234	218	416	52.4
	2	451	237	224	468	47.8
	3	474	211	208	485	42.8
	4	483	192	187	492	38.0
	5	484	157	149	496	30.0
	Nose	487	116	112	497	22.5
3	1	367	146	140	426	32.8
	2	440	131	129	472	27.3
	3	470	135	132	484	27.2
	4	493	129	129	493	26.1
	5	494	123	123	494	24.8
	Nose	494	99	99	494	20.0

Table 5. Drivers who can successfully see freeway vehicle while they merge from below left.

Geometric Type	Ramp Control Point	Vertical Plane Vision	Horizontal Plane Vision	Clear Vision (both planes)	Number of Opportunities	Drivers Able to See Clearly (percent)
1	1	402	301	391	414	70.2
	2	458	299	291	467	62.3
	3	474	259	252	484	52.0
	4	486	221	219	493	44.4
	5	492	191	190	496	38.3
	Nose	493	136	136	495	27.4
2	1	418	251	243	430	56.5
	2	454	215	209	467	44.7
	3	476	211	208	488	42.6
	4	489	189	187	498	37.5
	5	495	169	169	500	33.8
	Nose	493	108	108	498	21.6
3	1	417	150	145	424	34.6
	2	459	135	134	464	28.8
	3	479	123	121	482	25.1
	4	490	127	127	491	25.8
	5	492	108	108	494	21.8
	Nose	492	89	89	495	17.9

drivers can see the freeway vehicle clearly (that is, in both the vertical and the horizontal fields of vision) in geometric type 1 than in type 2 and in type 2 than in type 3. This would seem reasonable because a longer length of freeway segment is intercepted by the projection of the ramp dilemma zone in type 1 than in type 2. The same statement can be made for the comparison of types 2 and 3. It is evident, when these geometric types are examined, that the more nearly parallel the ramp and the freeway are, the less will be the likelihood that any particular ramp driver is able to see the freeway vehicle in the dilemma zone. Vision in the vertical plane in type 3, however, appears to be of slightly better quality than that in either type 1 or 2, especially near the ramp nose. This is probably due to closer proximity of the ramp pavement to the freeway pavement for a longer distance of the ramp dilemma zone.

Left On-Ramp to Freeway From Below—Table 5 gives the number of ramp drivers that are able to see in the vertical plane, in the horizontal plane, and in both planes when they merge from ramps below the freeway. The same basic generalizations may be made as for the ramp approach from above the freeway. Again, quality in the combined horizontal and vertical visual aspects deteriorates progressively from type 1 to type 3. Type 1 offers the most opportunities for clear vision, type 2 offers fewer, and type 3 offers the least number of opportunities for clear vision of freeway vehicles. There appears to be no discernible difference among the 3 geometric types for the ramp driver to see the freeway vehicle in the vertical plane.

Right On-Ramp to Freeway From Above—The result of the simulation run for merges on the right ramp from above are given in Table 6. In all 3 geometric types, only the horizontal vision angle was examined. It was assumed that a normal driver should be able to look over the driver's side doorsill in the vertical plane and experience no difficulty in observing a freeway vehicle on the roadway beneath him. Again, as in the left ramp merges, geometric type 1 appeared to offer the ramp driver the best quality of vision of the freeway; type 2 was somewhat inferior to type 1, and type 3 seemed to be the least adequate.

Right On-Ramp to Freeway From Below—Table 7 gives the results of the simulation run for the right merges from below. The major difference between this merging situation and the previous one is that the ability of the driver to see the freeway vehicle in the vertical plane may be impaired by the top of the driver's side door. Therefore, for this analysis, in addition to the vision in the horizontal plane, the vision in the vertical plane was also examined. Vertical vision in types 2 and 3 is about the same as that in type 1, although for all 3 types the number of drivers able to see the freeway vehicle was very high. This indicates that impairment of vertical vision is probably not a major problem for right-ramp merges from below. Vision quality in the horizontal plane was best for type 1, diminished for type 2, and lowest for type 3. Because of this, the quality of clear vision for the combined vision in the horizontal and vertical planes appeared to follow the trend of the horizontal vision quality.

Ramp Vehicle Location

For all geometric types, quality of clear vision decreases as the ramp vehicle approaches the ramp nose. This qualitative decrease is due to the length of the freeway segment scanned by the driver of the ramp vehicle as it moves in a lateral direction toward the freeway. Therefore, the driver's ability to detect a freeway vehicle was reduced accordingly because there would be less chance of that vehicle being in the shorter space.

Vision in Outside Rearview Mirror During Right Merge From Above

To fully examine the vision quality for the right merge from above, we tested vision in the outside rearview mirror in both horizontal and vertical planes to determine whether it might have a significant effect on merge vision. Table 8 gives data that indicate that rear horizontal vision appeared to be significant, expecially for type 3 geometrics. However, for all 3 types, rear vertical vision was negligible. Therefore, it is obvious that along the ramp dilemma zone, for a right merge from above, the outside rearview mirror is of little use in the visual detection of a freeway vehicle because the driver is almost never able to see that vehicle in the verticle rear vision plane.

Table 6. Drivers who can successfully see freeway vehicle while they merge from above right.

Geometric Type	Ramp Control Point	Horizontal Plane Vision	Number of Opportunities	Drivers Able to See Freeway Vehicle (percent)
1	1	340	401	84.7
	2	324	457	40.8
	3	302	479	63.0
	4	276	489	56.4
	5	232	494	46.9
	Nose	165	496	33.2
2	1	257	416	61.7
	2	274	468	58.5
	3	251	485	51.7
	4	221	492	44.9
	5	203	496	40.9
	Nose	145	497	29.1
3	1	187	426	43.8
	2	155	472	32.8
	3	174	484	35.9
	4	152	493	30.8
	5	156	494	31.5
	Nose	113	494	22.8

Table 7. Drivers who can successfully see freeway vehicle while they merge from below right.

Geometric Type	Ramp Control Point	Vertical Plane Vision	Horizontal Plane Vision	Clear Vision (both planes)	Number of Opportunities	Drivers Able to See Clearly (percent)
1	1	392	358	340	414	82.1
	2	444	342	323	467	69.1
	3	460	306	291	484	60.1
	4	470	269	259	493	52.5
	5	478	221	213	496	42.9
	Nose	478	176	175	495	35.3
2	1	409	292	278	430	64.6
	2	444	266	253	467	54.1
	3	466	253	243	488	49.7
	4	477	213	205	498	41.1
	5	481	205	196	500	39.2
	Nose	481	138	134	498	26.9
3	1	407	175	167	424	39.3
	2	449	156	151	464	32.5
	3	467	148	141	482	29.2
	4	475	152	144	491	29.3
	5	478	135	129	494	26.1
	Nose	479	112	109	495	22.0

Table 8. Additional drivers who can successfully see freeway vehicle in rearview mirror while they merge from above right.

Geometric Type	Ramp Control Point	Rear Vertical Plane	Rear Horizontal Plane	Rear Vision (both planes)
1	1	0	6	0
	2	0	25	0
	3	0	48	0
	4	0	67	0
	5	0	98	0
	Nose	4	189	4
2	1	0	31	0
	2	0	66	0
	3	0	99	0
	4	1	110	1
	5	2	157	2
	Nose	8	219	8
3	1	0	123	0
	2	0	179	0
	3	0	188	0
	4	0	202	0
	5	1	219	1
	Nose	6	257	6

Table 9. Interference of vehicle cockpit structure with horizontal vision of freeway vehicle.

Geometric Type and Merge	Ramp Control Point	Times Cockpit Prevents Vision: Number	Times Cockpit Prevents Vision: Percent	Number of Opportunities
1, left above	1	0	0	401
	2	0	0	457
	3	0	0	479
	4	1	0.2	489
	5	7	1.4	494
	Nose	47	9.5	496
2, left above	1	0	0	416
	2	3	0.6	468
	3	13	2.7	485
	4	18	3.7	492
	5	33	6.6	496
	Nose	97	19.5	497
3, left above	1	20	4.7	426
	2	74	15.7	472
	3	90	18.6	484
	4	75	15.2	493
	5	95	19.2	494
	Nose	146	29.6	494
1, left below	1	0	0	414
	2	0	0	467
	3	0	0	484
	4	1	0.2	493
	5	5	1.0	496
	Nose	41	8.3	495
2, left below	1	0	0	430
	2	1	0.2	467
	3	8	1.6	488
	4	10	2.0	498
	5	24	4.8	500
	Nose	77	15.4	498
3, left below	1	36	8.5	424
	2	67	14.4	464
	3	84	17.4	482
	4	69	14.0	491
	5	106	21.5	494
	Nose	153	31.0	495

Cockpit Interference in Left Merge

A separate evaluation was made to determine how the structural framework of the ramp vehicle's cockpit interferes with the ramp driver's horizontal vision in left merging. Table 9 gives the number of drivers whose horizontal vision is interfered with by the cockpit structural framework.

The cockpit interference with horizontal vision is most when the ramp and the freeway alignments are converging circular curves of the same sense (type 3 horizontal geometry). This is due to the very nearly parallel nature of the ramp and freeway roadways through the dilemma zone area. That parallel nature causes the respective offset distances to be less at every ramp station, which in turn reduces the length of freeway that the ramp driver is able to scan. The freeway vehicle of concern, therefore, has a greater chance of being outside the maximum visual scan allowed by the cockpit, and that situation would make it impossible for the ramp driver to see the freeway vehicle because of the cockpit framework. For the same reasons, cockpit interference with vision in the horizontal plane is apparently greater for type 3 than for types 1 and 2.

Comparison of Left and Right Merge Vision

Because this study attempted to establish whether there is a difference between a driver's ability to see the freeway traffic from left ramps and right ramps in mirror-image situations, a statistical test was conducted to examine significant differences between the left and right merge vision. The description of the procedure used in conducting the analysis is beyond the scope of this paper. However, the results of the statistical tests clearly indicated that there is a significant difference in the ramp driver's ability to see while moving through the dilemma zone of a left ramp compared to his ability to see while moving through the dilemma zone of a mirror-image right ramp. In 5 of the 6 types of ramp geometrics examined, the hypothesis that the visual quality of right ramps is superior to left ramps was accepted at the 0.90 level of confidence. The visual quality of types 1 and 2 right ramps to the freeway from above and type 1 right ramps to the freeway from below is significantly better than the visual quality of the corresponding left ramps at the 0.95 level of confidence.

Only for type 3 ramps to the freeway from below did there appear to be no significant difference between the left and right ramps; thus, they function about equally for either type of approach. However, ramps of this type show the lowest average percentage of clear vision of all the 6 types.

In general, z-scores were less for the ramp stations closer to the nose than for ramp stations farther upstream. This indicates that sideward visual quality decreases for all ramps as the ramp driver gets closer to the ramp nose. However, there are still significant differences in visual performance between right and left ramps for the geometric types considered in this study.

CONCLUSIONS

1. The porposed model provides information about the distribution of successful observations of a freeway vehicle by a ramp driver attempting to merge behind that vehicle from either a left or a right on-ramp. The model permits analysis of any on-ramp individually for visual quality by comparing visual successes at selected points along the ramp. Alternative on-ramp designs can also be compared to aid in the selection of the ramp type that will optimize ramp driver vision and enhance safety.

2. It is apparent for all ramps that the closer to the ramp nose the ramp driver is before he can look at the freeway the less chance he has of being able to see the freeway vehicle. Therefore, if guardrails or other obstructions prevent the driver from seeing freeway vehicles until he is close to the nose, the probability that he will not see the critical vehicle is increased. His decision time is also shortened as he moves closer to the ramp nose. The model results show that cockpit interference during left merges increases close to the ramp nose, and vision in the left side rearview mirror during

right merges is almost nonexistent at this point. These observations further justify the assumption that a proper merge dilemma zone of an on-ramp lies upstream from the ramp nose. Therefore, it is apparent that ramps with open dilemma zones in which obstructions are absent at points 200 to 300 ft upstream from the ramp nose are most conducive to safe merges.

3. On the basis of statistical tests conducted on the model results, at the 0.90 level of confidence the hypothesis that left on-ramps are inferior to right on-ramps because there are more hindrances to clear vision associated with left on-ramps is accepted for all ramp types except type 3 ramps to the freeway from below. In addition, the same hypothesis is accepted at the 0.95 level of confidence for types 1 and 2 from above and type 1 from below.

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SIMULATION OF CROSS-MEDIAN CRASHES

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This paper reports the results of a simulation study of vehicles crossing the median of a 6-lane expressway at angles of 10 and 50 deg. The simulation model allowed automobiles to be randomly released to cross the median at a constant speed and to encroach into opposing lanes. No provision was made for the driver to brake or regain control of the vehicle. Opposing vehicles were generated by the negative exponential distribution. The effects of the following variables on probability of crash and average impact speed were studied: crossing angle, lane volume, speed of opposing vehicles, speed of crossing vehicles; median width, perception-reaction time, and skid resistance. It was found that both probability of crash and average impact speed are most strongly related to crossing angle, median width, and speed of the crossing vehicle. Crash probability and impact speed are more fundamentally dependent on the time required for the vehicle to cross the median. An evaluation of the simulation model and a discussion of the applicability of the results to real traffic conditions are given.

•IN AN earlier paper (1), evidence was presented to show that cross-the-median crashes are rare in occurrence but severe in terms of extent of personal injuries. It was found in a study of Atlanta's police reports that during 1968 cross-the-median crashes, which accounted for only 4 percent of the expressway accidents, resulted in 54 percent of the expressway fatalities. A similar review of the police reports for 1970 revealed that only 1.9 percent of the expressway accidents involved one or more vehicles that actually crossed the median into opposing lanes; yet, these crashes resulted in 37.2 percent of the expressway deaths.

To attempt to better understand the nature of cross-the-median crashes and to evaluate the factors that contribute to their occurrence and severity, Wright et al. (1) developed a computer simulation model to describe vehicles that cross the median on a 4-lane expressway and proceed into the path of vehicles in opposing lanes. The principal shortcoming of that model was that it considered only vehicles that cross an expressway median at a right angle. Clearly, vehicles rarely encroach into an expressway median at angles even approaching 90 deg. Hutchinson and Kennedy (2) found that encroachment angles were typically 25 deg or less, and the average encroachment angle was about 11 deg.

This paper reports the results of a simulation study that considers the more realistic but complex case of vehicles crossing an expressway median at an acute angle.

COMPUTER MODEL

The computer program was written in ALGOL language and processed on a Burroughs B5500 computer. The model described a 6-lane, 2-way expressway with an unprotected center median. Some of the specific features of the model are as follows:

1. The highway had three 12-ft lanes in each direction and a variable median width.
2. A crossing car was released at a random instant at the far median edge and permitted to cross the median at a constant speed. Each vehicle thus released encroached

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into the opposing traffic lanes, and no provision was made in the program for the driver to brake or recover control of the car.

3. Oncoming vehicles in each of the opposing lanes were generated by the negative exponential distribution and, for a given run, were assumed to be traveling at equal constant speeds.

4. A crash was defined as occurring when the crossing vehicle and one of the opposing vehicles attempted to occupy the same space, called the collision zone, at the same time. For each lane, the collision zone was a rhombus resulting from the angular projection of 6.5 ft, the assumed vehicle widths (Fig. 1).

5. For a given run, a constant perception-reaction time was used; and after a delay equal to this value, the driver of the opposing vehicle was allowed to apply his brakes and attempt to skid to a stop. Neither vehicle was allowed to change direction of travel.

6. The time the collision zone was occupied by a vehicle was computed by considering the length of vehicle (assumed to be 17.5 ft), the length of the collision zone, and the vehicle speed. When the first opposing vehicle in lane 1 passed the collision zone before the crossing vehicle reached it, an additional vehicle was generated for lane 1 and checked for possible collision with the crossing vehicle. This process was repeated until all vehicles that might have had an opportunity to collide with the crossing vehicle were considered. When no crash occurred in lane 1, checks were made for lane 2 and similarly for lane 3.

7. The effect of crossing angle was tested by making 2 series of computer runs, one each for angles of 50 and 10 deg. A run consisted of 500 independent trials. For a given run, we assumed that lane volume, vehicle speeds, median width, skid resistance, and reaction time were constant. For each crossing angle, a total of 768 runs was made consisting of all possible combinations of the following variables:

Notation	Source of Variation	Amount
B	Lane volume, vehicles per hour	400, 800, 1,200, 1,600
F	Speed of opposing vehicles, mph	50, 60, 70
A	Speed of crossing vehicles, mph	20, 30, 40, 50
C	Median width, ft	10, 20, 30, 40
E	Perception-reaction time, sec	0.50, 1.0
D	Skid resistance	0.50, 0.65

Replicate runs were made for the 10-deg crossing angle series to facilitate analyses of variance. Thus, the simulation reported here consists of a total of 2,304 runs comprising more than 1 million independent trials.

8. The computer program kept account of the number of crashes and the speed of impact and computed the probability of crash and the average speed of opposing vehicle at time of impact.

The computer output data were used to plot graphs to illustrate the effect of the main variables on probability of a crash and average impact speed. These graphs show the likelihood of an out-of-control crossing vehicle colliding with opposing vehicles and provide a measure of crash severity in terms of average speed of the opposing vehicle at the moment of impact.

PROBABILITY OF CRASH

Crash probabilities were found to be most strongly influenced by encroachment angle, median width, and speed of the crossing vehicle. The probabilities were also dependent on lane volume, skid resistance, and, to a lesser degree, speed of the opposing vehicle and perception-reaction time.

Effect of Encroachment Angle

The effect of the main variables on the probability of crash, averaged over all possible observations, is shown in Figure 2. From a study of these graphs, it is apparent

Figure 1. Simulated roadway and vehicles.

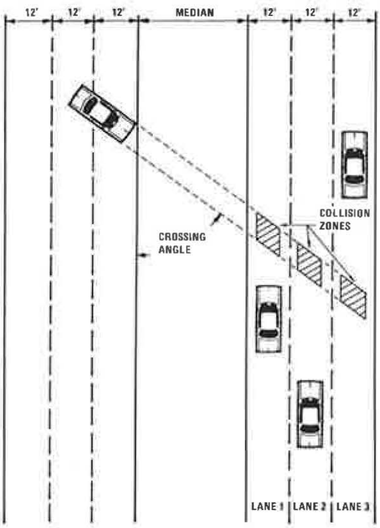


Figure 2. Average effect of main variables on probability of crash.

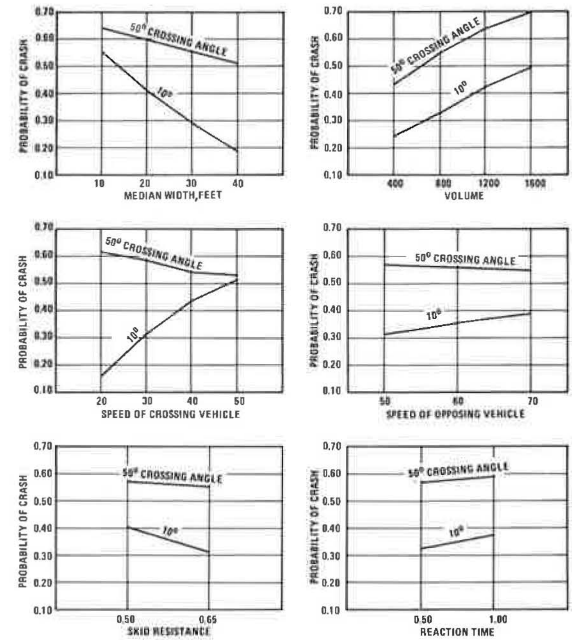


Figure 3. Average effect of median width on probability of crash for 10-deg crossing angle and various crossing speeds.

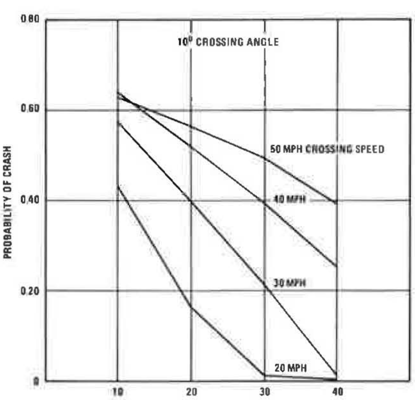


Figure 4. Average effect of lane volume on probability of crash for 10-deg crossing angle and various median widths.

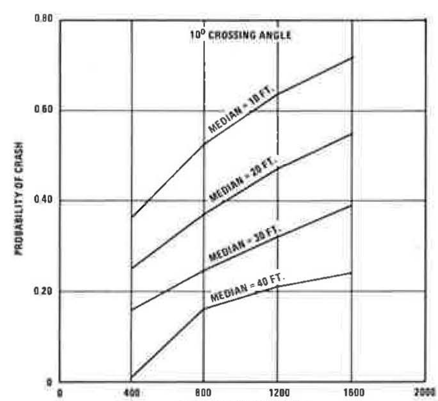


Table 1. Effect of skid resistance and reaction time on probability of crash and average impact speed.

	0.50-Sec Reaction Time		1.0-Sec Reaction Time	
Item	From	To	From	To
Skid resistance	0.50	0.65	0.50	0.65
Probability	0.387	0.290	0.427	0.338
Avg impact speed	20.81	16.07	24.19	20.10

Note: 10-deg crossing angle.

that the crossing angle is a major determinant of the probability of crash. In every instance, the 50-deg crossing angle results in larger crash probabilities. These results derive from the fact that large crossing angles are accompanied by shorter crossing times, and thus there is less opportunity for the driver of the opposing vehicle to brake and avoid a collision. Except when traffic volumes are light, crash probabilities were found to be generally greater than 0.50 when the crossing angle is 50 deg. Much smaller probabilities were found for the more common 10-deg encroachment angle.

Effect of Median Width

For a 10-deg crossing angle, crash probabilities are most significantly affected by the median width and the crossing speed. These effects, which are shown by Figure 3, indicate the value of providing wide median widths for expressways or, alternatively, of providing median barriers.

Effect of Lane Volume

As expected, probability of crash increases substantially with lane volume, generally more than doubling as the volume is increased from 400 to 1,600 vehicles per hour (Fig. 4).

Effects of Skid Resistance and Perception-Reaction Time

Variation in the skid resistance from 0.5 to 0.65 has little effect on the probability of a crash (Fig. 2 and Table 1). As expected, increases in skid resistance caused a decrease in crash probabilities. These figures show that the perception-reaction time had only a slight effect on the probability of a crash. Predictably, small perception-reaction times were accompanied by small crash probabilities.

AVERAGE IMPACT SPEED

Speeds of the opposing vehicles at the time of impact were also strongly dependent on encroachment angle, median width, speed of the crossing vehicle, and speed of the opposing vehicle. For the relatively small ranges chosen for skid resistance and reaction time, there was generally small variation in average impact speed. Lane volumes had little effect on impact speeds.

Effect of Encroachment Angle

Figure 5 shows the effect of the main variables on impact speed, averaged over all possible observations. These figures indicate that the encroachment angle strongly influences the average speed of the opposing vehicle at time of impact. The simulation results indicated that, for the 50-deg encroachment angle, impact speeds were generally high, averaging only about 10 mph less than the original speed of travel. Impact speeds greater than 50 mph were common for the 50-deg crossing angle. Much lower impact speeds were noted for the 10-deg encroachment angle.

Effect of Median Width

Figure 6 shows that for a 10-deg crossing angle average impact speeds decrease sharply with increase in median width. As expected, these speeds were found to be dependent on the original speed of the vehicle. For an assumed original speed of 70 mph, average impact speeds generally higher than 40 mph were found for a 10-ft median width. In comparison, average impact speeds less than 15 mph were found when a 40-ft median width was employed.

Increasing the median width produced relatively small decreases in average impact speed when an encroachment angle of 50-deg was assumed. This finding dramatizes the fact that in cross-the-median crashes involving large crossing angles there is usually little time for a driver to react and slow his vehicle to a tolerable impact speed by braking.

Figure 5. Average effect of main variables on average impact speed of opposing vehicle.

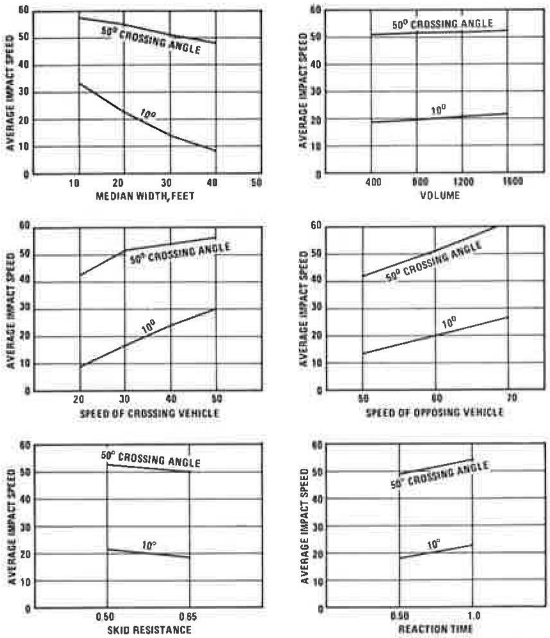


Figure 6. Effect of median width on average impact speed of opposing vehicle for 10-deg crossing angle and various original speeds.

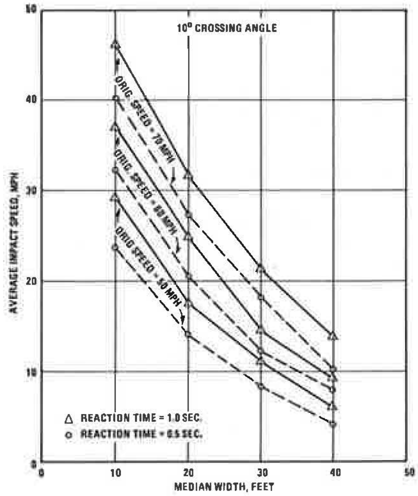


Figure 7. Effect of speed of crossing vehicle on average impact speed of opposing vehicle for 10-deg crossing angle and various median widths.

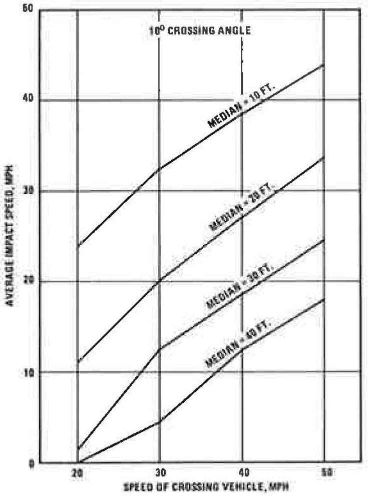


Table 2. Variables affecting probability of crash for 10-deg crossing angle.

Source of Variation	DF	Mean Square	Source of Variation	DF	Mean Square
C	3	106,964.74	CF	6	260.88
A	3	98,629.48	BF	6	246.43
B	3	47,224.13	AE	3	151.11
D	1	33,310.81	EF	2	140.57
F	2	7,557.17	BE	3	124.44
E	1	7,223.67	CD	3	117.15
AC	9	3,883.42	DF	2	104.17
AB	9	1,997.40	DE	1	59.30
BC	9	1,405.81	CE	3	12.11
AD	3	757.61			
BD	3	617.95			
AF	6	450.05	Experimental error	768	4.22

Notes: Significance is at 1 percent level. Three-way interactions did not provide important relationships. Four and higher order interactions were not investigated. A = speed of crossing vehicle; B = volume; C = medium width; D = skid resistance; E = reaction time; and F = speed of opposing vehicle.

Table 3. Variables affecting speed of impact for 10-deg crossing angle.

Source of Variation	DF	Mean Square	Source of Variation	DF	Mean Square
C	3	49,529.99	AC	9	144.76
A	3	31,068.34	AD	3	85.34
F	2	18,380.96	CD	3	110.89
D	1	7,494.68	EF	2	47.40
E	1	5,279.69	DE	1	40.41
B	3	115.44	BC	9	11.87
CF	6	600.67	AB	9	8.47
AF	6	502.24	BD	3	4.39
DF	2	297.01	BF	6	3.55
AE	3	281.12			
CE	3	159.85	Experimental error	768	0.49

Notes: Significance is at 1 percent level. Three-way interactions did not provide important relationships. Four and higher order interactions were not investigated. A = speed of crossing vehicle; B = volume; C = medium width; D = skid resistance; E = reaction time; and F = speed of opposing vehicle.

Effect of Speed of Crossing Vehicle

As shown in Figure 7, the higher the speed of a crossing vehicle is, the more likely it will be struck by a vehicle moving at a high rate of speed. Figure 7 shows that barriers are required in narrow medians of high-speed roadways to prevent vehicles from encroaching into the path of vehicles traveling at a high rate of speed. Although the advantages of wide medians in reducing impact speeds is apparent, barriers may still be justified in wide medians if the violent and sometimes fatal crashes that result from out-of-control vehicles crossing at a large angle and high speed are to be prevented.

Effect of Skid Resistance

Skid resistance had a significant but unimportant effect on the average impact speed. Increasing the skid resistance from 0.50 to 0.65 reduced the impact speed an average of only about 4 mph (Table 1).

Effect of Perception-Reaction Time

Data given in Table 1 indicate that, with small perception-reaction times, a greater portion of the available time may be devoted to braking and that lower average impact speeds result. For the small range of perception-reaction times used, however, this effect was relatively unimportant.

ANALYSIS OF FINDINGS

An analysis of variance was made for the data pertaining to a 10-deg encroachment angle to evaluate the effect of the 6 main variables on the probability of a crash and the average impact speed. Each analysis of variance was designed as a completely factorial experiment considering the 6 variables as being fixed.

There was strong a priori belief that the main variables would significantly affect probability of crash and average impact speed. Indeed, this was the basis on which the main variables were chosen. The principal value of the analysis of variance was, therefore, to rank the main variables in order of significance and to provide a quantitative evaluation of the effect of interactions. The results of the analyses of variance are given in Tables 2 and 3.

All of the main variables and 2-way interactions had a significant effect on the probability of a crash. Similarly, all of the main variables and all but one of the 2-way interactions significantly affected the average speed of impact.

An analysis of the simulation results indicated that a fundamental variable affecting both the crash probability and the impact speed was the crossing time, defined as the time required for a vehicle to cross the median and encroach into the path of a vehicle in lane 1.

$$CT = [(M + 2.75)/\sin \alpha]/CS$$

where

CT = crossing time, sec;
CS = crossing speed, fps;
M = median width, ft; and
α = crossing angle, deg.

The greater is the crossing time, the longer has the driver of the opposing vehicle to decrease the speed of his vehicle or bring it to a stop. Significantly, both crash probability and impact speed were most strongly related to median width, crossing speed, and angle of crossing; these 3 variables determine the time required to cross the median.

The relation of probability of crash and impact speed to crossing time is shown in Figures 8 and 9.

Figure 8. Effect of median crossing time on probability of crash.

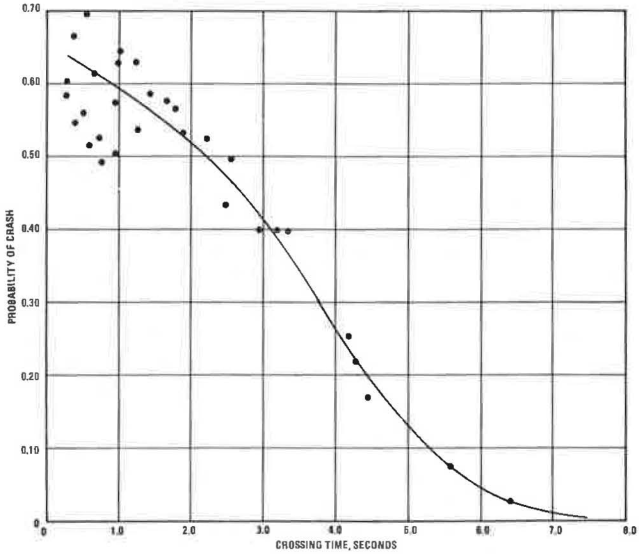


Figure 9. Effect of median crossing time on average impact speed.

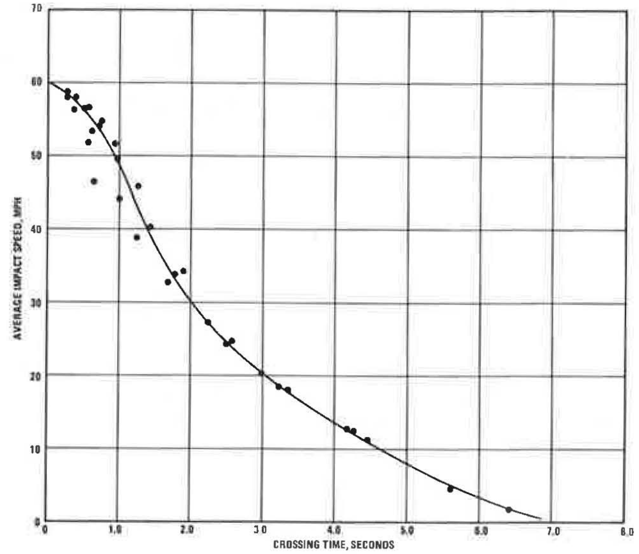
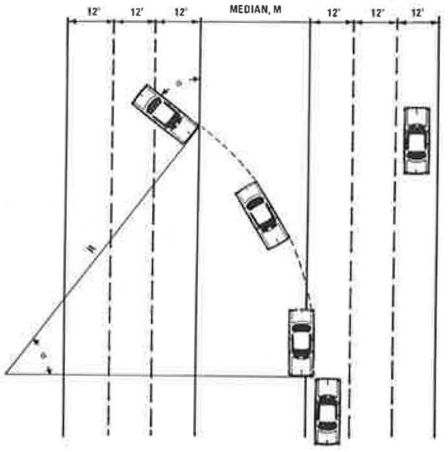


Figure 10. Relations for vehicle that avoids encroaching into opposing lane.



EVALUATION OF THE COMPUTER MODEL

Because the model does not permit the crossing driver to stop or regain control, it clearly does not accurately simulate the behavior of all median-encroaching vehicles. It simulates instead those vehicles that actually trespass into opposing traffic lanes. One should, therefore, not directly apply the crash probabilities to all median-encroaching vehicles because a certain percentage of those vehicles are brought under control and redirected to the original direction of travel. This percentage would be expected to increase as the crossing time increases, e.g., in cases involving wide medians, slow crossing speeds, or small crossing angles, or all of these. Under such conditions, the probability that a median-encroaching vehicle will cross into opposing lanes and crash is low indeed, probably even lower than shown by the figures.

On the other hand, the conditions that result in a short crossing time (e.g., narrow median width, high crossing speed, or large crossing angle, or all of these) make it unlikely that the encroaching vehicle can be brought under control. For these conditions, the generally high probabilities reported would be expected to be fairly representative of probabilities that median-encroaching vehicles crash.

For a given encroaching speed S and an assumed coefficient of side friction f, there is a corresponding minimum turning radius R for an encroaching vehicle (3). The relationship is

$$S = 5.5\sqrt{f \times R/2}$$

If f = 0.50, the turning radii for various encroaching speeds are as follows:

Encroaching Speed (mph)	R_{min} (ft)
20	52
30	120
40	210
50	330

Suppose the driver does not brake but begins to redirect the vehicle at the instant it encroaches into the median. The vehicle can be assumed to follow a circular path of radius R_{min}. Figure 10 shows that

$$\cos \alpha = (R_{min} - M - 2.75)/R_{min}$$

From this equation, the minimum width of median, M_{min}, to prevent a vehicle from encroaching into an opposing lane, can be computed for various values of crossing angle and crossing speed as follows:

Crossing Speed (mph)	10 deg	30 deg	50 deg
20	-2	4	16
30	0	13	40
40	1	25	72
50	2	41	115

The driver of a vehicle that crosses the median at a large angle and high rate of speed is unlikely to avoid encroaching into oncoming lanes. On the other hand, a reasonably alert and proficient driver operating a mechanically sound vehicle that encroaches into the median at a small angle and a slow rate of speed would almost certainly avoid encroaching into opposing lanes.

It should be remembered that the simulation provided only for automobiles, each assumed to be 6.5 ft in width and 17.5 ft in length. Because trucks describe a larger collision zone, are longer, and occupy the collision zone for a longer period, one would expect real-life crash probabilities to be higher than those reported if the traffic stream contains a significant percentage of trucks.

Finally, the reaction times chosen for this study (0.5 and 1.0 sec) assume near-ideal conditions. These reaction times do not allow for a distracted, intoxicated, or day-dreaming driver or one with an unusually large perception-reaction time because of age or physical disability. Any factor that would result in a perception-reaction time greater than those used would tend to increase the probability of a crash and the impact speed.

SUMMARY AND CONCLUSIONS

The simulation model used in this study described vehicles that randomly cross an unprotected median of a 6-lane expressway and encroach into opposing traffic lanes. The vehicles crossed at angles of 10 and 50 deg to the roadway centerline, and no provision was made for the crossing driver to brake or change the direction of his vehicle. After a delay equal to his perception-reaction time, the opposing driver was allowed to apply his brakes and attempt to skid to a stop without changing direction of travel. Significant findings from the research are given below.

1. Both probability of crash and average impact speed are most strongly related to encroachment angle, median width, and speed of the crossing vehicle. More fundamentally, crash probability and impact speed are principally dependent on the median-crossing time.
2. Crash probabilities were generally greater than 0.50 for a 50-deg crossing angle and ranged from about 0.20 to 0.55 for a 10-deg crossing angle. As expected, high crash probabilities were recorded for narrow median widths, high lane volumes, and fast vehicle speeds.
3. Average impact speeds for the opposing vehicles were high for the 50-deg crossing angle ranging typically from about 40 to 60 mph. For the 10-deg crossing angle, average impact speeds ranged from about 10 to 30 mph. Not surprisingly, low impact speeds resulted when wide median widths were employed. Other factors that resulted in large crossing times tended to produce low impact speeds.
4. Cross-median accidents occur rarely but constitute an important part of the severe and fatal accident problem. More extensive employment of wide medians or median barriers or both will be required if the number of these tragic crashes is to be reduced.

ACKNOWLEDGMENT

This research was supported by an institutional grant from the National Science Foundation, and that assistance is gratefully acknowledged.

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REDUCING IMBALANCE OF INTERSECTING FREEWAYS BY ON-FREEWAY CONTROL

Roy C. Loutzenheiser, Texas Transportation Institute, Texas A&M University; and Donald M. Henderson, Wilbur Smith and Associates

Because of a heavy traffic demand on merging ramps during peak periods and a lane imbalance at the merge areas, an interchange of Interstate 610 and Interstate 10 in Houston has ramp queues that frequently extend onto upstream freeways. A morning and an evening study was made to evaluate the effects of on-freeway control as a means of improving the merge operation. Standard maintenance procedures were used to close the outside freeway lane upstream of the merge area for a short period of time at the beginning of the merge congestion. Based on vehicle counts and average vehicle data that were collected before and during the closure study, a comparison of changes in flow rates and in total delay was made. Because accident data were not available, only nonincident days were analyzed. During morning and evening closures, the flow rates through the merge areas did not change; however, the morning closure caused a 9 percent increase in delay, whereas the evening closure reduced delay by 2 percent. The average flow rate on the 2 open freeway lanes during closure was about 1,650 vehicles per hour per lane. Implementation of positive lane closure was too time-consuming for a short-term closure. Therefore, the method of positive closure used in this study was not the optimal solution for this interchange. Further studies using other methods of on-freeway control have been recommended.

•MANY major interchanges are experiencing severe traffic congestion during peak-period operation. The problem is usually associated with a merge area where there is a lane imbalance and where there is a heavy traffic demand on the merging ramp during the early part of the peak period. At the beginning of the peak period, the ramp demand exceeds the ramp capacity at the merge area while the upstream freeway demand is below capacity. The heavy ramp demand results in reduced traffic flow in the merging lanes and usually in the total merging area. Hazardous operation may cause a breakdown in the merge area operation, and ramp queues that extend into the crossing freeway may result.

The premise is that better interchange operation is achieved by traffic control or minor geometric modifications. Control of modifications would permit a balanced lane operation at the merge points when traffic demands approaching the merge area exceed the merge capacity. Three general solutions to the problem are add a lane in the merge area, reduce the number of lanes approaching the merge area, and reduce the traffic demand approaching the merge area. Any one, or combination, of these 3 approaches can provide relief to the problem.

The success of adding lane in the merge area depends on the length of the added lane. If the lane is extended to an exit ramp, the solution should be valid. If it is lengthened a few hunderd feet and then dropped, the problem will not be completely resolved. However, the prolonged lane may result in an improved situation because of a longer tapered design and additional sight distance.

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Reducing the number of lanes approaching the merge area to the same number as those leaving the merge area will not directly increase the capacity of the freeway, but it should improve the flow characteristics. That, in turn, would improve the safety of the area and would result in improved capacity because of the reduction of incidents. The method of closing a lane on a freeway has been investigated by several people. In general, two approaches have been used:

1. Positive closure of a lane—lane closure is usually needed only during the peak period, and therefore permanant closure is considered impractical. To manually close a lane on a daily basis is impractical; however, there is not a good method at present for effective automatic positive closure of a lane.
2. Voluntary lane closure—The most practical way to effect a lane closure is to use signs and signals that can be activated when needed. Several devices that could be used in an installation of this type are available. Some examples of these are advanced warning signs with fixed or variable messages and a red X and green arrow display. The obvious problem is that motorists may not obey the control devices if they know there is little danger of being involved in an accident or being fined by the police.

Reduction of the demand on approaches to the merge area may, in some instances, be feasible by using traffic control devices on the approaches. Ramp metering, a successful traffic control system for entrances to the freeway, could be used on the interchange roadways. The objective here is to reduce demand for very short time periods or to coordinate the flows approaching the merge area. Bulk metering would be used.

Each day miles of freeway lanes are being closed for the purpose of maintenance and construction. Kermode and Myyra (1) developed a procedure that will enable field personnel to schedule lane closures at a time when these closures will cause the least inconvenience to the motorists. Lee (2) discussed special procedures to be used during nighttime work.

Studies are being made to determine the best procedure to implement reversible lanes for unbalanced flow. Waight (3) described how a 2-lane, reversible tunnel was built in San Francisco to increase the traffic flow on 2 parallel roadways, 2 lanes each way. A system of movable, flexible barriers and changeable signs has been developed to control the traffic during the rush hours. DeRose (4) studied the operation of a reversible center-lane traffic system on an undivided roadway. The signing consisted of lane control signals (red X and green arrow) and NO LEFT TURN signs.

Forbes and Gervais (5) made a study of the effectiveness of symbols for lane control signals. Their studies showed that the red X and green arrow were meaningful in providing proper control. Hoack, Madsen, and Newman (6) found that ramp control on a 2-lane, high-speed, high-volume entrance ramp in the Los Angeles area reduced the holiday congestion at a major interchange.

Several interchanges in Houston experience serious breakdown in traffic operation during the morning and evening peak periods. The reduction is caused by a heavy traffic demand on the ramps prior to the heavy freeway demand. After some preliminary investigation, the interchange at Interstate 610 and Interstate 10 west was selected as a location where part of the preceding premise could be studied. The study design was based on reducing the number of lanes approaching the merge area by temporarily closing the outside freeway lane by positive means.

With the cooperation and assistance of the Texas Highway Department, the Texas Transportation Institute proposed that a traffic control system be designed, installed, operated, and evaluated at this major interchange to improve traffic operations and safety during peak periods. This paper presents some of the more important findings in the study; details of the study can be found in another report (7).

STUDY PROCEDURES

Description of Sites

Traffic operations at the I-610 and I-10 interchange frequently break down because of heavy traffic demands on the ramps. During the morning peak period, the queue on the ramp extending from I-10 eastbound to I-610 southbound (AM site) frequently backs onto the I-10 eastbound freeway lanes. In a similar manner, the queue on the ramp extending from I-610 northbound to I-10 westbound (PM site) backs onto I-610 northbound lanes. The 2 sites are shown in Figure 1. Each site has 3

Figure 1. Study sites.

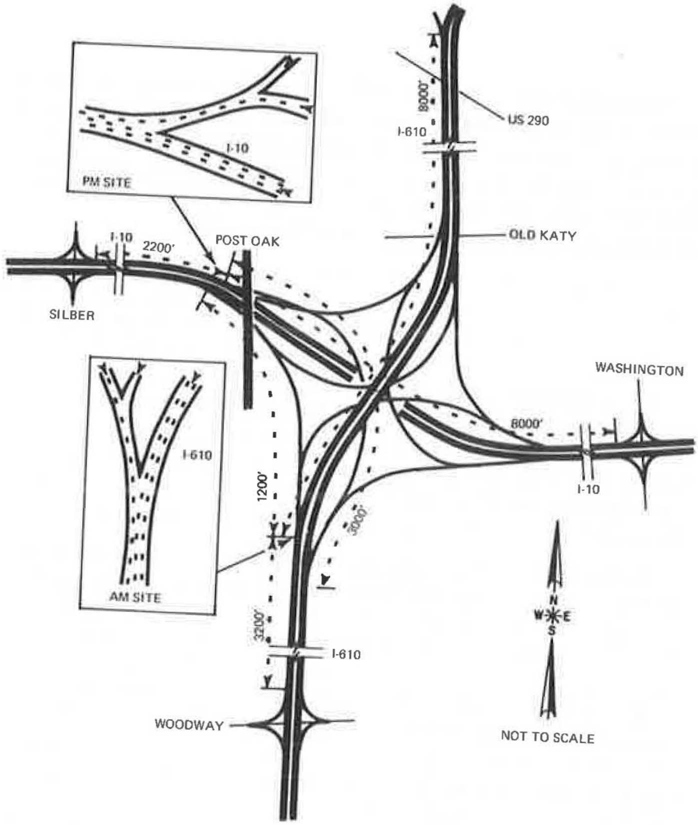
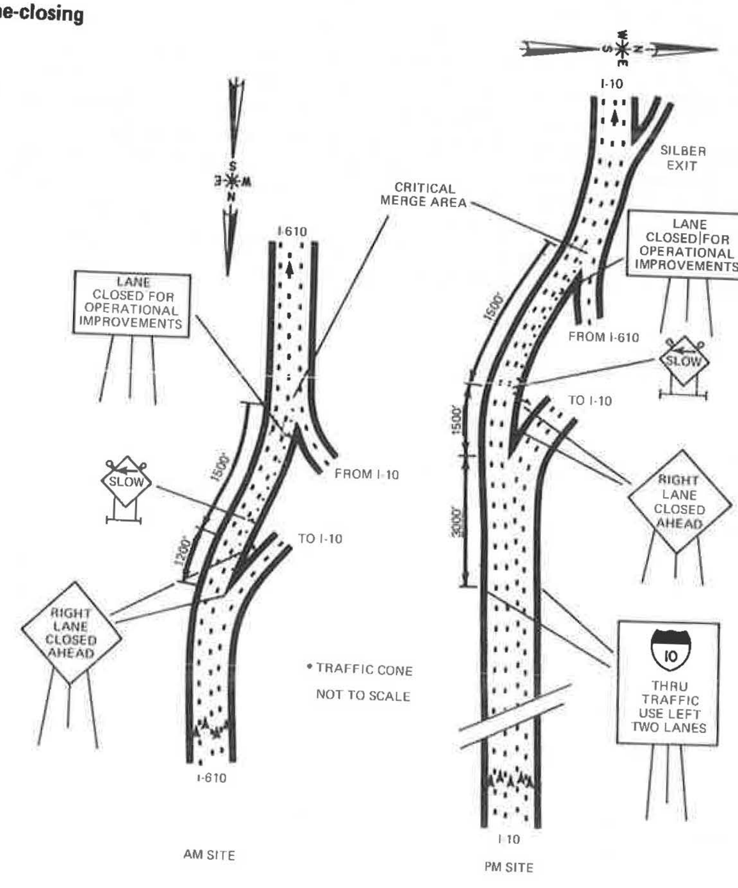


Figure 2. Sign placement in lane-closing operation.



upstream freeway lanes and 2 ramp lanes merging into 4 downstream freeway lanes. Interstate 610 is elevated above I-10.

At the AM site the I-10 eastbound to I-610 southbound ramp is about 1,200 ft in length and provides limited storage capacity. The ramp has an uphill grade that reduces visibility of the merge area. The upgrade also increases the recovery time that is required when ramp vehicle speeds are reduced. The first downstream exit ramp on I-610 southbound is about 3,200 ft from the merge point, and the last upstream entrance ramp is about 8,000 ft from the merge point.

Drivers using the I-610 northbound to I-10 westbound ramp at the PM site can see both the merge area and the input freeway lanes because of the downgrade approaching the merge area. The ramp is long (about 3,000 ft) and has some storage capacity. Because the I-10 freeway is depressed at the interchange, the freeway driver is unable to see the merge area until he is close to it. The first exit ramp downstream on I-10 is about 2,200 ft from the merge point, and the last entrance ramp upstream is about 8,000 ft from the merge point. About 6,600 ft downstream of the merge point, the freeway lanes on I-10 are reduced from four to three.

Preliminary Traffic Evaluation Before Closure

Visual observation indicated that the ramp backup at the AM site contributes to the daily reduced flow on I-10 eastbound. Flow on I-10 eastbound improved once vehicles were beyond the interchange. Peak flow through the merge area usually occurred around 7:45 a.m.; however, there was a significant secondary peak around 8:15 a.m. The inside lane of the ramp, which merges with the outside freeway lane, carried a small percentage of the ramp movement. Consequently, the outside ramp lane had frequent queuing that extended onto I-10 eastbound and formed shock waves. Traffic flow on I-610 southbound, upstream of the merge area, remained at a high level of service and was impeded only when an accident occurred.

The evening peak period at the merge on I-10 occurred around 5:20 p.m. However, the ramp usually had a significant increase in flow around 5:00 p.m., which was 5 to 10 min earlier than the initial freeway buildup. The ramp had reduced operations during this time, and a queue frequently extended onto I-610 northbound. This queuing caused reduced flow on the outside lane and frequently the 2 outside lanes on I-610 northbound. The reduced freeway operation caused a slowly moving queue on the I-610 freeway for several miles upstream. The upstream I-10 freeway demand remained below capacity until congestion occurred in the merge area. Because of the freeway lane drop or an incident, downstream operation on I-10 westbound occasionally backed into the merge area and caused the interchange operation to break down prematurely.

Lane Closure Technique

For a pilot study of on-freeway control, 2 of the 3 control methods mentioned earlier were not feasible. Because the AM site is on an embankment and the PM site is in a depression, construction of an additional freeway lane would be costly. Reducing the traffic demand approaching the merge area is unreasonable because one objective is to increase the ramp flow.

Inasmuch as both sites have an input freeway demand of less than capacity, a reduction in the number of lanes approaching the merge area was considered feasible. Closing the outside freeway lane by positive means was recommended and was accomplished in the same manner used by maintenance forces to block a lane. Advanced signing alerted motorists of the closure, and traffic cones and signs effected the physical closure. A special trailer-mounted sign with flashing beacons was used to enhance the safety of operations. The actual lengths of closure were about 1,500 ft, which included a 750-ft taper. Additional warning signs were placed about 3,000 ft upstream of the beginning of closure for the PM site. Figure 2 shows the location of closure signs.

On the first day of closure, the outside lane at each site was closed from 6:40 to 7:40 a.m. and from 4:35 to 5:30 p.m. The capacity of the remaining 2 lanes of the freeway proved inadequate, so closure was reduced to less than 1 hour on the second day. Closure time was further reduced to less than 40 min for the remainder of the study. Beginning closure time and the total closure time for each day were based on real-time field decisions and the results of the previous closures. The study was conducted on weekdays from the evening of June 7 to the evening of June 23, 1971. Because of rain, the closure was cancelled on 2 evenings (June 18 and 22). The morning closures were conducted as planned.

ANALYSIS OF DATA

Several months prior to the closure study, preliminary data were collected to establish a basis for comparison. For this report, only data collected during nonincident periods were used in the analyses. Usable vehicle counts before closure were obtained on only 3 days because of the limitation of available manpower. An "average" vehicle study was made in 2 vehicles during a period of several weeks. In this study, each driver was instructed to follow a predetermined route as a typical driver. The second person in each vehicle recorded travel times to various predetermined stations, queue formations, and a general subjective evaluation of the interchange operation. Current accident data for this interchange were not available or were insufficient.

During the closure study, similar data collection was made. In addition, some observations were made from an airplane during 2 mornings and 4 evenings. So that a meaningful comparison could be made, data collected during an incident were not used. Also, data from the first 2 days of closure for each site were not used so that more "typical" findings could be provided. During the study, data for 8 mornings and 5 evenings were analyzed.

Analysis of Traffic Flow

The initial closure for each site lasted about 1 hour and caused upstream freeway queues of several miles in length and resulted in significant delay. Some reasons for this queuing were insufficient advance notice to the public, extended time required to manually close the lane, and a larger than expected reduction in capacity by the closed outside lane. After the first 2 days of closure, intervals of closures varied from 15 to 37 min.

Typical volumes for a nonincident day before closure and a day during closure are given in Table 1. In general, the total output volumes were the same. The reduced upstream freeway flow was compensated by the increased ramp flow. Figure 3 shows the percentage of ramp flow to total flow through the merge area for 5-min intervals before and during closure. These percentages were based on typical, nonincident days.

AM Site—The total volume through the merge area was about 6,850 vehicles (1,710 vehicles per lane) between 7:00 and 8:00 a.m. for both before and during the study. During the period from 6:45 to 8:15 a.m., the volume increased by more than 200 vehicles during the closure. Between 7:00 and 8:00 a.m., the freeway input volume decreased from 4,150 to 3,800 vehicles, while the ramp input volume increased from 2,700 to 3,100 vehicles. During the closure, the 2 open freeway lanes had an average flow rate of more than 1,650 vehicles per hour per lane. Table 2 gives a summary of the daily closure time and merge operation.

Because the number of vehicles leaving the freeway at the upstream exit ramp on I-610 increased by 40 vehicles, it was assumed that the closure caused only minor diversion. The downstream exit ramp on I-610 has minor effect on the operation in the merge area because of the light flow rate. During the closure, the exit ramp volume decreased by 25 vehicles.

Without closure, the AM site ramp usually carried about 40 percent of the total flow through the merge area; however, this percentage decreased during the peak half-hour (Fig. 3). With closure, the percentage increased to about 50 percent during the peak half-hour. Preliminary counts before closure indicated that the left lane of the ramp was used by less than 20 percent of the ramp traffic flow during the peak flow, except for a short period of time. During the closure, it was anticipated that more vehicles would use the left lane; however, counts showed little change in the percentage of usage. The percentages are shown in Figure 4. The apparent reason for this lack of utilization was inability of ramp drivers to see the closed outside freeway lane or the merge area. Some form of information sign was needed on the ramp.

Table 1. Volume counts made at I-10 and I-610 interchange before and during closure.

Location of Vehicles When Counted	Number of Lanes	Vehicles at AM Site				Vehicles at PM Site			
		7:00-8:00		6:45-8:15		5:00-6:00		4:45-6:15	
		Before	During	Before	During	Before	During	Before	During
On freeway	3	4,159	3,781	5,882	5,459	3,945	3,673	5,706	5,327
Entering at ramp	2	2,686	3,113	3,729	4,367	2,707	3,000	4,108	4,535
Total	4	6,845	6,894	9,611	9,826	6,652	6,673	9,814	9,862
Leaving at downstream exit	1	189	164	305	278	719	718	1,002	1,057
Leaving at upstream exit	2 and 3	1,956	1,992	2,791	2,524	2,156	2,302	3,665	3,165

Table 2. AM site volume counts downstream of merge.

Date	Vehicles 7:00-8:00	Vehicles 6:45-8:15	Flow Rate on I-610 During Actual Closure	Closure Time	Closure Duration (min)
10-13-70	6,510	9,655		Before	
1-13-71	6,835	9,733		Before	
1-14-71	6,827	9,612		Before	
6-08-71	—[a]	—[a]		6:40-7:40	60
6-09-71	6,529	9,500	1,703	7:02-8:00	58
6-10-71	6,894	9,826	1,698	7:16-7:51	36
6-11-71	6,450	9,700	1,728	7:16-7:53	37
6-14-71	6,751	9,818	1,760	7:22-7:52	30
6-15-71	6,604	9,579	1,632	7:30-7:55	25
6-16-71	6,747	9,829	1,639	7:30-7:57	27
6-17-71	6,718	9,746	1,608	7:29-7:52	23
6-18-71	6,538	9,424	1,633	7:29-7:51	22
6-21-71[b]	6,463	9,237	1,706	7:29-7:45	16
6-22-71	6,668	9,547	1,562	7:29-7:47	18
6-23-71[c]	6,748	9,546	1,690	7:30-7:45	15

[a]No counts.
[b]Stalled car on ramp at merge from 7:21 to 8:10.
[c]Minor accident on ramp at merge from 7:45 to 7:48 and then moved to shoulder.

Figure 3. Ramp flow as a percentage of total flow through the merge area on a typical nonincident day.

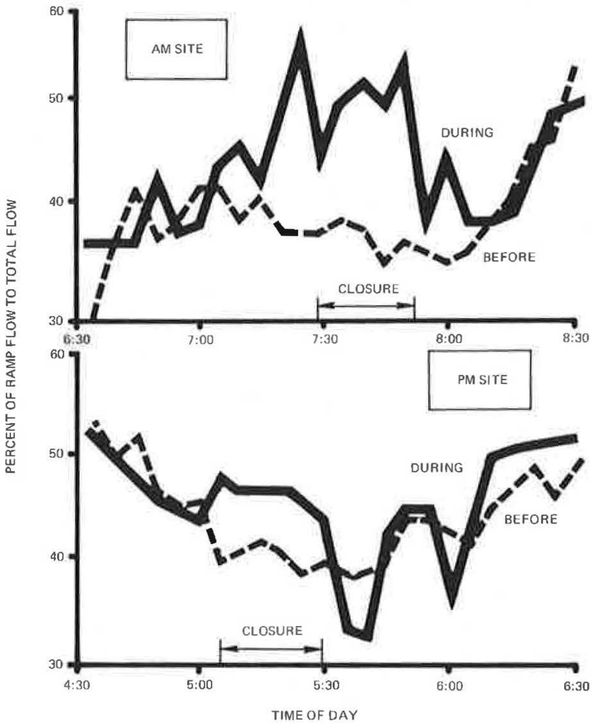
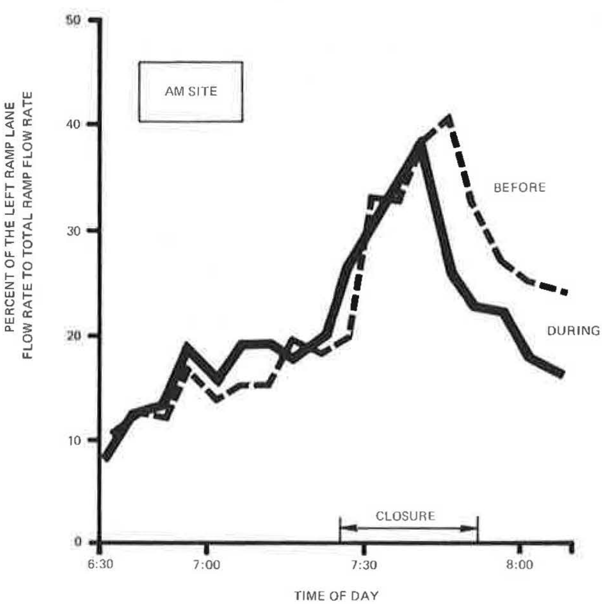


Figure 4. Lane distribution of traffic flow from I-10 eastbound to I-610 southbound ramp before and during closure.



PM Site—The total volume through the merge area was about 6,650 vehicles between 5:00 and 6:00 p.m. (1,660 vehicles per lane) before and during the study (Table 1). Between 4:45 and 6:15 p.m., the total volume increased by fewer than 50 vehicles during the study. The freeway input between 5:00 and 6:00 p.m. decreased from 3,950 to 3,650 vehicles, while the ramp input volume increased from 2,700 to 3,000 vehicles. During the closure, the 2 open freeway lanes had an average flow rate of about 1,650 vehicles per lane. Table 3 gives a summary of the daily closure time and merge operation.

During the lane closure, the upstream exit ramp volume on I-10, between 5:00 and 6:00 p.m., increased by 150 vehicles (about 7 percent), and some queues were observed on this exit ramp. Apparently some motorists were diverting from I-10 upstream of the closure. The downstream exit ramp on I-10 had little change in volume. It had been anticipated that there would be a decrease in volume at this ramp. Occasionally poor operation at this downstream exit ramp continued to generate shock waves that affected the merge area.

Prior to the closure study at the PM site, the percentage of ramp flow to total merge flow varied from 50 percent before the peak hour to about 40 percent during the peak hour. Figure 3 shows that the closure permitted the ramp flow to remain slightly higher than usual. As anticipated, the flow on the left ramp lane increased. This increase was due to the clear view of the merge area and lane closure. Figure 5 shows the change in percentage of left lane flow to total ramp flow with time.

Average Vehicle Study

A parameter used in determining the effectiveness of on-freeway lane closure is the change in total delay at the interchange as calculated from the average vehicle study. A successful study is one in which total delay is reduced. The anticipated effect in lane closure is improvement of operation at the merge, downstream from the merge, on the ramp, and on the crossing freeway. Reduction in operation upstream of the merge on the freeway is expected. For this study, analysis was made for $1\frac{1}{2}$ hours at each site (6:45-8:15 a.m. and 4:45-6:15 p.m.) to include most delayed effects of closure.

As expected, the AM site had an increase in delay upstream of the merge on I-610 because of the reduction in lanes. There are no good alternate routes for I-610 southbound traffic. A reduction in the increased delay was expected once closure procedures were improved and motorists became familiar with the closure; however, this reduction did not occur. The I-10 eastbound flow also had an increase in delay. The conclusion is that other factors, such as upstream entrance ramps, were causing delay on I-10 in addition to the extended queue on the I-610 exit ramp. It was previously determined (Fig. 4) that the left lane of the exit ramp was not fully used as anticipated and, therefore, the ramp queue was only partially reduced. Delays to the motorists on the ramp and downstream of the merge area decreased slightly. Total delay for the AM site increased by 132 vehicle-hours or by 9 percent. A summary of the data is given in Table 4 (7, 8). The different subsystems used in analysis of the delay are shown in Figure 6.

Closure at the PM site was successful in decreasing delay on the crossing freeway (I-610) and on the ramp from I-610 to I-10 westbound. A queue on this ramp began to form prior to closure but dissipated after the closure was initiated. The delay increase on I-10 upstream of the merge was significant, and some diversion was taking place near the end of the 13-day study. Diverting motorists probably found less delay on alternate routes. The freeway flow immediately downstream of the merge improved and had a reduction in delay. Farther downstream, where the I-10 freeway lanes decrease from 4 to 3, there was a slight increase in delay. Total delay for the PM site decreased by 23 vehicle-hours or by 2 percent (Table 4).

Operational Effects

Even though some public announcements were made before the study, it was apparent that the motorists were not prepared for the closure. Two accidents on the freeway occurred upstream of the merge and might have resulted from the extended queue formation. As previously mentioned, data on accidents were not available. Three stalled

Table 3. PM site volume counts downstream of merge.

Date	Vehicles 5:00-6:00	Vehicles 4:45-6:15	Flow Rate on I-10 During Actual Closure	Closure Time	Closure Duration (min)
10-13-71	6,501	9,615		Before	
1-13-71	6,635	9,810		Before	
1-14-71	6,610	9,875		Before	
6-07-71	5,478	8,447	1,661	4:35-5:50	75
6-08-71[a]	6,435	9,424	1,695	4:47-5:29	42
6-09-71[b]	6,057	9,345	1,692	4:52-5:20	28
6-10-71[c]	6,293	9,465	1,531	5:03-5:25	22
6-11-71	6,487	9,524	1,555	4:57-5:31	34
6-14-71[d]	6,594	9,712	1,729	4:55-5:27	32
6-15-71	6,255	9,287	1,694	5:05-5:24	19
6-16-71	6,219	9,314	1,709	5:05-5:30	25
6-17-71[e]	6,347	9,435	1,566	5:05-5:29	24
6-18-71	—[f]	—[f]	—[f]		
6-21-71	6,673	9,862	1,615	5:05-5:36	31
6-22-71	—[f]	—[f]	—[f]		
6-23-71	6,103	9,238	1,648	5:09-5:29	20

[a]Minor accident on I-10 upstream from 5:12 to 5:25, and stall downstream on I-10 at 5:45.
[b]Stalls downstream on I-10 from 5:00 to 5:23, 5:32 to 5:54, and 5:53 to 5:59.
[c]Stall on I-10 in closure area from 5:13 to 5:34, and car smoking on shoulder of ramp from 5:36 to 5:50.
[d]Minor accident on I-10 upstream from 5:23 to 5:37, and stall downstream on I-10 from 5:20 to 5:35.
[e]Minor accident downstream on I-10 from 4:50 to 5:05, and stall on ramp in merge area from 5:43 to 5:46.
[f]Rain.

Figure 5. Lane distribution of traffic flow from I-610 northbound to I-10 westbound ramp before and during closure.

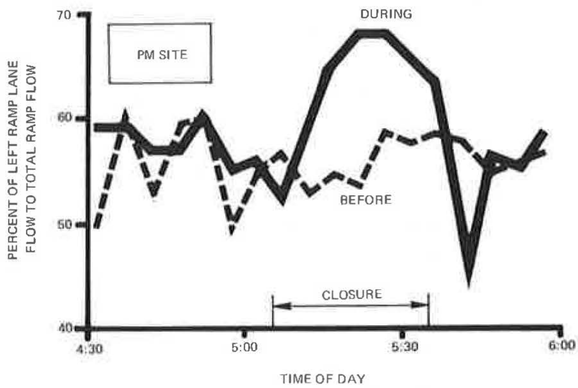
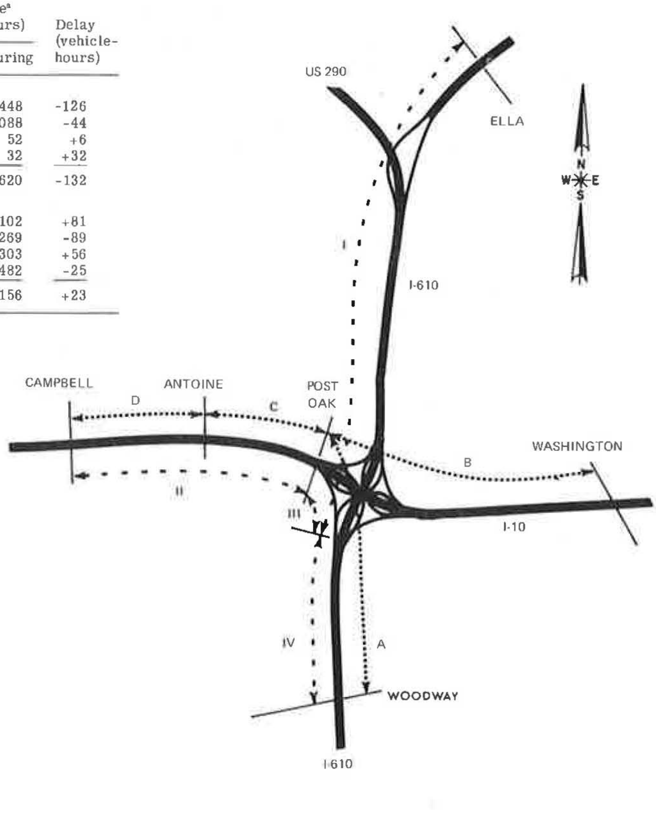


Table 4. Change in minimum average speed and total travel time due to lane closure based on average vehicle study.

Section	Length of Section (miles)	Minimum Avg Speed (mph) Before	Minimum Avg Speed (mph) During	Avg Total Travel Time[a] (vehicle-hours) Before	Avg Total Travel Time[a] (vehicle-hours) During	Delay (vehicle-hours)
AM site						
Ella to merge	3.0	46	19	322	448	-126
Campbell to Post Oak	3.0	20	20	1,044	1,088	-44
Post Oak to merge	0.6	27	32	58	52	+6
Merge to Woodway	0.8	32	34	64	32	+32
Total				1,488	1,620	-132
PM site						
Woodway to Post Oak	1.1	12	23	183	102	+81
Washington to Post Oak	1.5	30	12	180	269	-89
Post Oak to Antoine	1.1	18	23	359	303	+56
Antoine to Campbell	1.8	29	29	457	482	-25
Total				1,179	1,156	+23

[a]Time period is 6:45 to 8:15 a.m. for AM site and 4:45 to 6:14 p.m. for PM site.

Figure 6. Subsystems used in average vehicle study.



vehicles in the merge area and downstream of the merge could not be attributed to the effects of the closure.

After a more desirable time period for closure was established, the queue formation on the upstream freeway was limited to $1\frac{1}{2}$ miles. The queue movement was usually stop-and-go. When the closure was removed, the freeway queue usually dispersed within 15 min. The queue on the ramp from I-610 to I-10 at the PM site ramp did not extend onto the I-610 northbound freeway lanes. However, at the AM site the queue on the I-10 to I-610 ramp briefly extended onto the I-10 eastbound freeway lanes because of the poor utilization of the left ramp lane.

As previously mentioned, closure of the outside freeway lane was based on experience of previous closures and usually lasted between 15 and 30 min. The closure was initiated after a queue began to form on the ramp and after the input freeway flow started to increase. There were insufficient data to determine a flow parameter for initiation of closure; however, on most of the good operational days the lane was closed when the combined 5-min input flow (freeway and ramp) exceeded 600 vehicles.

The closed freeway lane was not reopened until the ramp queue was eliminated and until the freeway downstream of the merge was operating fairly well. Before the lane was opened, sufficient capacity in the merge area was needed to handle the increased freeway input.

As expected, there were some public complaints about the closure study. Motorists who usually traveled on an unobstructed freeway upstream of the merge complained about the reduced speeds and queue on the input freeway. However, those complaints were more than offset by compliments about the improved operations of the interchange. The motorists, who no longer encountered the stop-and-go flow on the crossing freeway, approved of the lane closure. Most comments, good or bad, were about the PM site.

SUMMARY OF FINDINGS

1. During the study at the AM site, the 7:00 to 8:00 a.m. volume for the I-610 freeway flow upstream of the merge area decreased from 4,150 to 3,800 vehicles, while the ramp volume increased from 2,700 to 3,100 vehicles. The flow rate on the 2 open freeway lanes during closure was 1,650 vehicles per hour per lane.
2. The motorists on the AM site ramp did not fully utilize the inside lane because they were unable to see the merge area. Some form of information sign was needed on the ramp to advise motorists of merge area operation.
3. The total delay for the interchange between 6:45 and 8:15 a.m. increased by 9 percent because of the continued poor operation on I-10 eastbound upstream of the I-610 exit ramp.
4. During the study at the PM site, the 5:00 to 6:00 p.m. volume for the I-10 freeway flow upstream of the merge area decreased from 3,950 to 3,650 vehicles, while the ramp volume increased from 2,700 to 3,000 vehicles. The flow rate on the 2 open freeway lanes during closure was 1,650 vehicles per hour per lane.
5. The ramp queues at the PM site were eliminated, which resulted in a 2 percent decrease in total delay at the interchange between 4:45 and 6:15 p.m.
6. Manual implementation of positive lane closure is too time-consuming and distracting to provide desirable on-freeway control.
7. A better operational solution may be obtained by geometric modifications for the AM site and automatic voluntary lane closure for the PM site. Better communications with the motorist about any changes are needed.
8. Public opinion supporting the lane closure was greater than that disapproving.
9. A solution to interchange congestion, caused by heavy merge flow rates arriving at different times within a peak period, is on-freeway control; however, further research is needed to determine when this control should be applied.

ACKNOWLEDGMENT

The opinions, findings, and conclusions expressed or implied in this report are those of the authors and not necessarily those of the Texas Highway Department or of the Federal Highway Administration.

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FEASIBILITY AND EVALUATION STUDY OF RESERVED FREEWAY LANES FOR BUSES AND CAR POOLS

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Alan M. Voorhees and Associates, Inc.; and
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The specific major objectives of this study were (a) to determine the feasibility of moving more people with fewer vehicles and of improving traffic operation on a given freeway by reserving one or more lanes for the exclusive use of buses and car pools during normal weekday peak periods and (b) to develop a plan for demonstration and evaluation of the operational effectiveness of the reserved-lane concept. Development of a demonstration and evaluation plan (the second objective) was contingent on demonstration of feasibility in the first phase of the study. Research objectives were pursued through the performance of the following project tasks: analysis of freeway types and reserved-lane rules to determine the general potential of the reserved-lane concept; selection of a specific freeway site for a detailed feasibility study; detailed analysis of traffic operations to determine the implications of the reserved-lane concept; analysis of user cost to compare reserved-lane operation with normal operation; analysis of modal choice to estimate potential shifts from low-occupancy vehicles to car pools and buses; investigation of the legislative and enforcement requirements involved in implementing the reserved-lane concept; assessment of requirements for a public information and education program to support concept implementation; and formulation of an implementation plan for demonstration of the reserved-lane concept. Results of the research indicate that the concept is basically sound and offers the potential benefit of reduced congestion at a given level of passenger demand.

•THOSE who are most articulate in their complaints about freeway traffic congestion and breakdown will have to admit when pressed that, aside from the 3 to 5 maximum flow hours each weekday, urban freeways operate pretty well. They accommodate a range of vehicle types—trucks, buses, and private cars—with varied trip purposes and widely diffused origins and destinations throughout the metropolitan area.

However, the slowdowns and stoppages that occur during those daily peak-volume periods, with which so many urban commuters are drearily familiar, are a problem. Some suggest the construction of more and more freeway mileage in urban areas. Transportation planning in this country during the past 10 years or so, however, has shown that there is an upper limit to freeway construction for social and political reasons if not for operational reasons. Moreover, even if those limitations did not exist and resources were unlimited, studies and forecasts of travel demand suggest that peak-period congestion would still be a daily occurrence.

It is unfortunate that peak-volume periods that feature the greatest person demand for person movement are accompanied by the lowest average person occupancy per automobile. This phenomenon has the effect of multiplying the demand for "moving steel" at a given level of person movement demand. It is caused by the predominance

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of the work trip during the peak hours, the travel purpose typically featuring the lowest average automobile occupancy. Potentially offsetting this, however, is the fact that work travel lends itself best to being served by public transportation, the most efficient form of urban person movement.

There has been considerable effort in recent years to increase the person-moving efficiency of urban highways by granting exclusive lane use to buses. There are a number of places where this has been accomplished, but in most places it cannot be done for the simple reason that the headways between the buses would be excessive, leaving the lane unused much of the time. For reasons of efficiency and public acceptance, it is necessary to fill up the spaces between the buses. Furthermore, if we are to truly maximize the person-moving capacity of a bus lane, the logical way to do it is to fill up those spaces with the most efficient use of the automobile for moving people—the car pool.

This idea is not new. Cherniack suggested it in 1963 (1). In 1968 the Federal Highway Administration encouraged highway departments to give it serious consideration (2). In 1970 the Institute of Transportation and Traffic Engineering developed a priority-lane model that analyzed the various strategies of reserving one or more lanes for buses and vehicles with more than "n" number of occupants (6). In 1971 the Alan M. Voorhees firm undertook a detailed feasibility study of reserving a freeway lane for buses and car pools for a specific facility (8). That study utilized the priority-lane model developed by ITTE and also examined other important feasibility aspects such as enforcement and public information.

The reason for the interest and involvement is that there is concern for taking direct action to implement a reserved lane for buses and car pools. The potential payoff appears to be too great and the need to get more efficiency out of the urban freeways is too urgent to allow a potentially viable scheme to remain in the discussion stage. Therefore, the Office of the Secretary of Transportation, the Urban Mass Transportation Administration, and the Federal Highway Administration jointly sponsored the research study reported in this paper.

OBJECTIVES OF THE STUDY

This research study was undertaken to ascertain whether the reserved-lane concept will accommodate the travel demand on urban highways and, at the same time, reduce vehicular congestion during peak traffic periods. Specific major objectives of the study were as follows:

1. To determine the feasibility of moving more people with fewer vehicles and of improving traffic operation on a given freeway by reserving one or more lanes for the exclusive use of buses and car pools during normal weekday peak periods and
2. To develop a plan for demonstration and evaluation of the operational effectiveness of the reserved-lane concept.

Development of a demonstration and evaluation plan (the second objective) was contingent on demonstration of feasibility in the first phase of the study. Research objectives were pursued through the performance of the following project tasks: (a) analysis of freeway types and reserved-lane rules to determine the general potential of the reserved-lane concept; (b) selection of a specific freeway site for a detailed feasibility study; (c) detailed analysis of traffic operations to determine the implications of the reserved-lane concept; (d) analysis of user costs to compare reserved-lane operation with normal operation; (e) analysis of modal choice to estimate potential shifts from low-occupancy vehicles to car pools and buses; (f) investigation of the legislative and enforcement requirements involved in implementing the reserved-lane concept; (g) assessment of requirements for a public information and education program to support concept implementation; and (h) formulation of an implementation plan for demonstration of the reserved-lane concept.

RESERVED-LANE CONCEPT

A mathematical model developed by the University of California for the Federal Highway Administration was used in the general analysis. This is a traffic-flow model that analyzes the effect of assigning any number of lanes on a freeway with as many as 5 directional lanes to vehicles with any specified number of occupants. It is capable of accepting any range of vehicle-occupancy distribution, speed-flow relation, and travel demand. It considers whether either a queuing or nonqueuing flow relation exists for the priority and nonpriority lanes and compares the resulting travel time for any strategy with the original travel time. The approach was to hypothesize initial freeway conditions under normal operations for varying demand-capacity conditions. Then, while passenger demand was held constant for each initial demand-capacity level considered, reasonable shifts of automobile passengers into buses and into car pools were projected.

Reserved-lane operation configurations were considered for 3, 4, and 5 freeway lanes in 1 direction. For each of those 3 configurations, 3 different levels of initial automobile occupancy were investigated, and 5 levels of peak vehicular demand were tested; volume-capacity ratios ranged from 1.05 to 1.25. Four hypothetical levels of bus occupancy—ranging from 0 to 15 percent of total passenger demand—were considered. Initial automobile-occupancy distribution was tested along with 3 hypothetical levels (5, 10, and 15 percent) of shift from low-occupancy automobiles to car pools. Results of the general flow analysis of the reserved-lane concept are shown in Figure 1.

The reserved-lane concept appears generally promising. Each freeway width for each initial occupancy level has at least 1 configuration with good potential. Among the conclusions are the following:

1. Wider freeways appear to provide greater flexibility in selection of the reserved-lane scheme;
2. Except for the 5-lane freeway with medium initial occupancy, reservation of 2 lanes for buses and car pools appears to have limited potential;
3. Four-lane freeways show good potential for reservation of 1 lane for vehicles with 3 or more occupants for all initial automobile-occupancy conditions considered;
4. Initial automobile-occupancy distribution is an important factor in determining the potential feasibility of the reserved-lane concept for a given freeway;
5. The most serious problem to be dealt with in application of the reserved-lane concept is congestion during the transition phase while people shift into car pools and buses, and this problem can be overcome by implementing the concept before severe traffic demands exist on the freeway.

SITE SELECTION

Analysis of a specific freeway representing as closely as possible the conditions existing in a large number of urban areas was considered the most effective approach to the study. Constraints on site selection were developed and applied to a list of 25 possible sites, acquired through recommendations from the Federal Highway Administration and selected professionals. The Memorial Shoreway (I-90) in Cleveland, Ohio, was chosen for detailed analysis.

The Memorial Shoreway is the backbone of a major transportation corridor serving the Cleveland metropolitan area on the south shore of Lake Erie. The freeway segment selected for the study site is located in Cuyahoga County and passes through parts of the cities of Cleveland and Euclid and the village of Bratenahl. The study section (about 12 miles in length) has 4 directional lanes, a high type of geometrics with fairly standard interchange designs, and good terminal conditions.

The area served by the freeway is diverse. Development consists primarily of office buildings and a large university complex in the central business district. To the east, I-90 borders on a major low-income pocket. Large industrial developments lie to the south, and a medium-income residential area is located to the north. In Lake County, I-90 passes through a high-income, low-density area. This mix of uses offered the opportunity to investigate the effects of assigning the reserved-lane concept on many different trip types.

TRAFFIC OPERATIONS

The operations analysis of the feasibility of the reserved-lane concept on the Memorial Shoreway included the following elements: collection of specialized data; analysis of freeway flow using a deterministic flow model; evaluation of the effect of accidents and other lane-blocking incidents and the effect of future traffic growth; evaluation of lane-selection criteria; detailed analysis of volume-capacity relations; investigation of safety implications; and development of a traffic control plan for reserved-lane operation.

Data Collection

Various types of specialized data were collected directly or were obtained from existing sources. Because very few detailed data were available concerning traffic operation on the freeway, substantial new data collection activities were required for the project. In addition, data relating to existing transit operations were supplied by the Cleveland Transit System, and transportation planning data needed were provided by the Northeast Ohio Area Coordinating Agency.

Freeway Flow

The mathematical model of freeway flow developed by the Institute of Transportation and Traffic Engineering of the University of California, Berkeley, was used as a tool in evaluating the feasibility of the reserved-lane concept. The model's output provided comparisons of freeway operations characteristics under normal operation and under reserved-lane operation; under the latter condition, 1 lane was reserved for buses and car pools having 3 or more occupants.

The approach used in applying the model was to exercise it for initial automobile occupancy and bus ridership conditions existing on the Shoreway and also for hypothetically shifted automobile occupancy and bus usage levels covering the full range of possible changes in travel behavior that might be induced by the initiation of the reserved-lane concept. Consideration was given to the effect of shifts on traffic volumes, passenger volumes, travel time, average speed, and total passenger-hours. Figure 2 shows the effect of shifts on total passenger-hours of travel. Anything greater than a combination of a 5 percent shift into buses and a 5 percent shift into car pools will reduce the total hours of passenger travel.

Effect of Capacity-Impairing Incidents

In the flow analysis given above, freeway operation was considered under ideal, incident-free, unimpaired capacity conditions. However, accidents and other capacity-impairing incidents frequently occur on freeways. Thus, it would be risky indeed not to ask the questions, How does the reserved-lane concept work on days when incidents occur? Will the concept still produce operational benefits when a capacity impairment occurs? Answers to these questions were pursued by using the freeway flow model to simulate the effects of capacity-impairing incidents.

The analysis of the effect of capacity-impairing incidents indicated that, if the reserved-lane concept works under ideal incident-free conditions, it will also produce operational improvements on days when accidents or other capacity-impairing incidents occur. Figure 3 shows the effect on passenger-hours of travel of a ½-hour lane blockage.

Effect of Future Traffic Growth

Another factor considered in evaluating the feasibility of the reserved-lane concept of operation was the effect of future traffic growth. Will increasing traffic demands negate the effectiveness of reserved-lane operation as compared with normal operation with no lane-usage regulations? To determine the answer, we made 2 analyses: (a) a general flow analysis in which the effect of various V/C ratios was examined and (b) a specific flow analysis in which traffic volumes on the Shoreway were projected 4 years

Figure 1. General feasibility of reserved-lane concept.

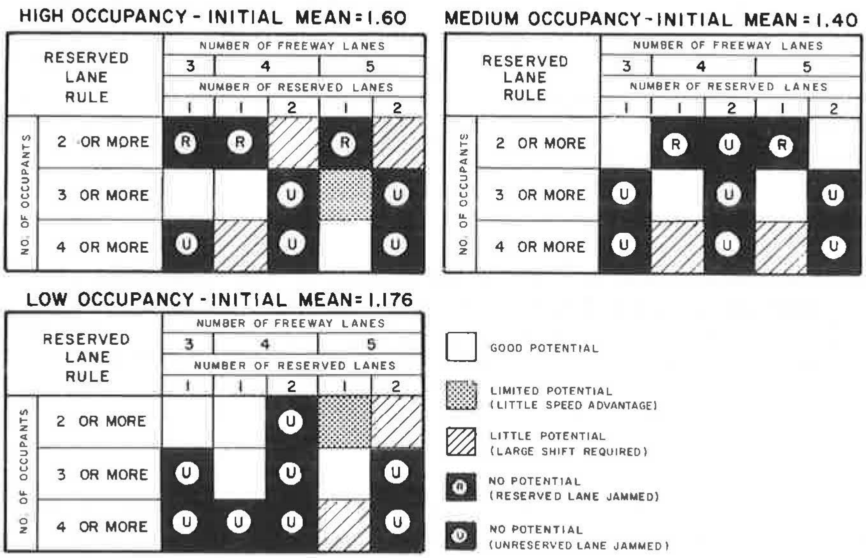


Figure 2. Effect of shifts on total inbound passenger-hours during 2½-hour commuting period in 1970.

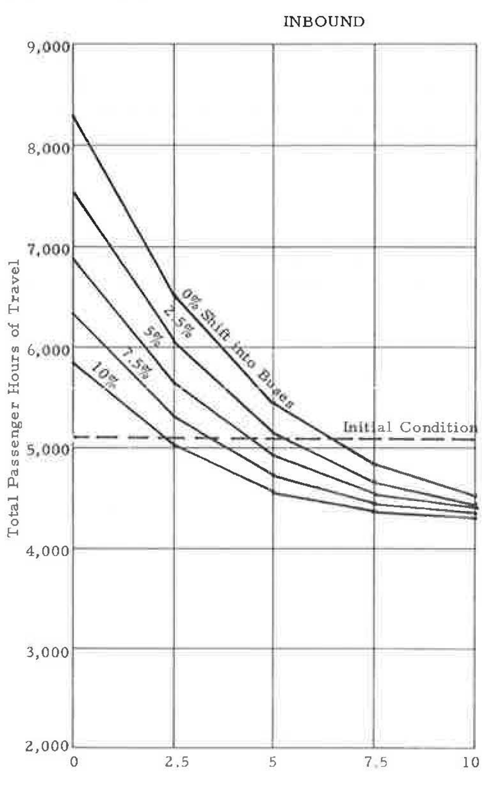
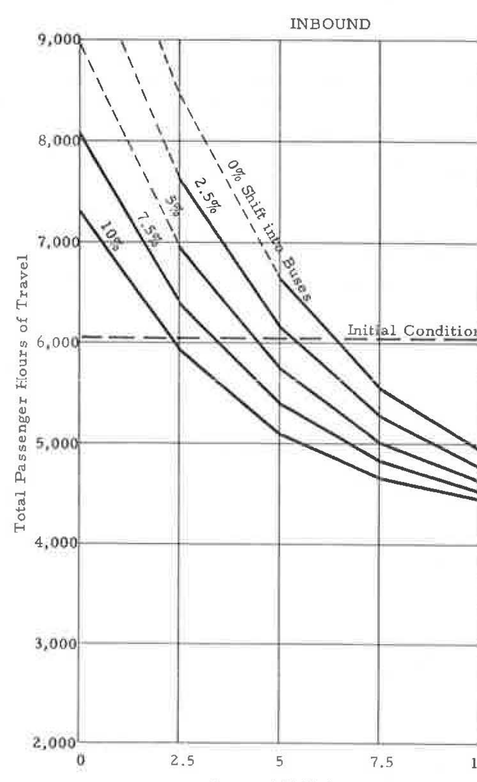


Figure 3. Effect of shifts on total inbound passenger-hours with ½-hour lane blockage.



into the future by using an annual growth rate of 2.5 percent. In both analyses, the freeway flow model was used to test the reserved-lane concept under initial and hypothetically shifted conditions of automobile occupancy and bus usage.

The general flow analysis showed that the break-even point in terms of user benefits occurs at approximately the same degree of shift for all volume-capacity ratios and that the lane-reservation scheme is flexible in terms of adapting to changing demand. This is shown in Figure 4. In other words, the flexibility of the reserved-lane concept can be sustained into the future as growth in travel demand occurs. It is important to note, however, that because of the transition problems it is better to introduce the reserved-lane operation at an early date when demands and congestion are still moderate. The critical period appears to be the initial condition prior to shifts into buses and car pools.

The specific flow analysis for the Shoreway indicated that, with a 5 percent shift into car pools and buses, unreserved-lane volumes are reduced nearly back to the level experienced prior to shifting, when no lane-usage regulations were in effect.

Lane-Selection Criteria

An analysis was performed to determine which lane on the freeway is the best one to use as the reserved lane. In the approach to this problem, a relatively simple analysis was made of the approximate total number of lane changes required on the freeway in order to distribute entering traffic across the 4 lanes and to distribute traffic across the lanes to the exit ramps. It was assumed that all entrance and exit ramps are on the right side.

The evaluation of lane-selection criteria showed conclusively that, on the Memorial Shoreway, the left lane should be the one used as the reserved lane. In addition to a series of logical reasons for selection of the left lane, the analysis indicated that the total amount of lane changing required to distribute freeway traffic across the lanes from and to entrance and exit ramps will be minimized if the left lane is reserved. This is shown in Figure 5. Other reasons for selecting the left lane include the following:

1. The left lane is traditionally occupied by high-speed traffic;
2. Marginal friction is lowest in the left lane because it has adjacent traffic flow only on one side;
3. The left lane is least affected by the turbulence created by merging and diverging traffic, assuming that ramps are on the right;
4. If the left lane is reserved, very short-distance trips will be discouraged from using the freeway because of the number of lane changes that would be required;
5. Slowly moving truck traffic traditionally keeps to the right (in some jurisdictions, this is a legal requirement);
6. All buses using the Shoreway operate on an express basis, and there is no need for them to travel on the right side after entering the freeway;
7. The development of a traffic control plan would be greatly simplified in that the left lane would be easier to sign; and
8. Enforcement of the reserved-lane rule is simplified if the left lane is used because nonqualified vehicles should never be in the reserved lane.

Capacity

The operational implications of the reserved-lane concept were further analyzed by the use of Highway Capacity Manual techniques. The elements included were analysis of main-line demand, capacity, and queuing; evaluation of distance required for weaving at the beginning and end points of reserved-lane operation; and analysis of service volumes at all merging and diverging points along the freeway. In addition, the effect of the reserved-lane concept on the person-carrying capacity of the freeway was evaluated.

Results of the demand-capacity analysis confirmed the findings of the freeway flow analysis performed with the mathematical model. During the transition state, prior to

Figure 4. Effect of increasing traffic demand on total passenger-hours under reserved-lane operation.

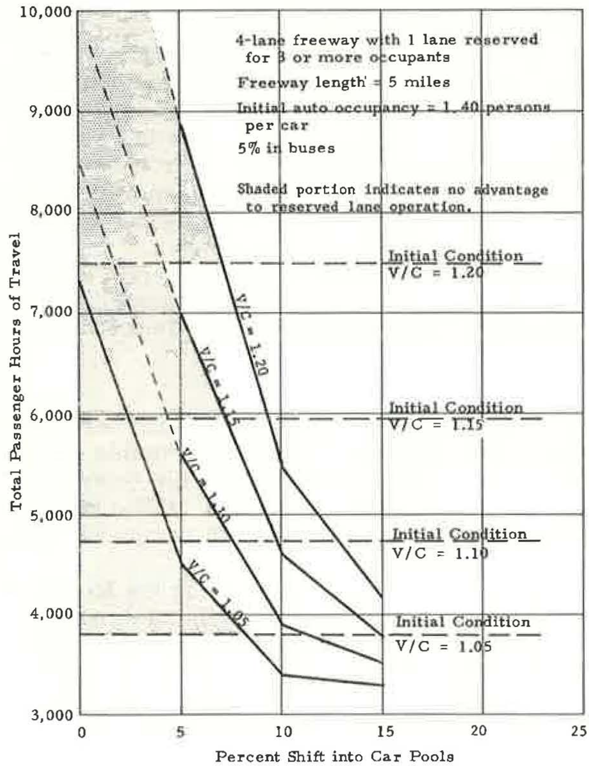
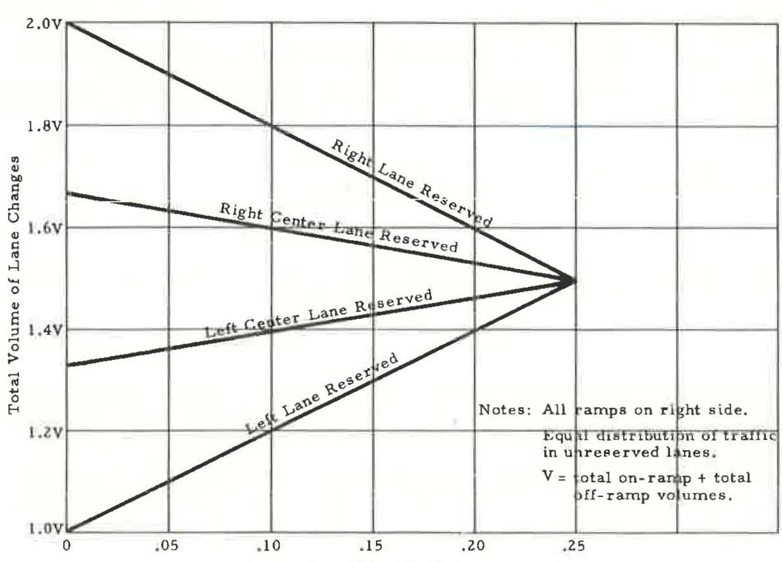


Figure 5. Effect of proportion of traffic in reserved lane on total lane changing required.



shifting into car pools and buses, congestion and queuing in the 3 unreserved lanes will be increased compared with normal operation. However, given the anticipated degree of shifting induced by reserved-lane operation, the traffic demand-capacity relations in the unreserved lanes will approximate existing normal conditions, and only a few minor localized points of congestion will remain. These are shown in Figure 6.

An evaluation of the person-carrying capacity of the freeway under reserved-lane operation showed that shifts in mode are required to increase the freeway's capability for people movement. If shifts occur greater than 6 percent into car pools and 6 percent into buses, then the person-carrying capacity will be higher under the reserved-lane concept than under normal operation. The reserved-lane scheme developed for the Shoreway has the potential of increasing the facility's person-carrying capacity by approximately 13 percent.

Safety Implications

Precise prediction of the influence of the reserved-lane concept on the overall safety of freeway operation was not possible. The only real test of whether the concept will significantly affect safety—either positively or negatively—is a full-scale trial conducted for a period of sufficient duration and under a broad enough variety of day-to-day conditions to allow reliable statistical comparisons of accident experience. However, it was apparent from the study that cogent arguments can be raised, both pro and con, concerning the prospective influence on overall safety of the reserved-lane concept. Positive factors include reduced vehicle volumes and subsequent exposure due to persons shifting to higher occupancy vehicles, smoother freeway operation due to reduced volumes, reduced total lane changing, and lower ramp merging volumes. Negative factors include the speed differential between the reserved and unreserved lanes and weaving at the reserved-lane terminal points. It is believed that many of the important influencing factors will tend to nullify each other, and no dramatic changes in collision frequency will be experienced. A full-scale demonstration is necessary to produce a reliable answer to the question of whether the reserved-lane concept will degrade, enhance, or leave unchanged the existing level of safety of freeway operation.

Traffic Control Plan

Three alternate methods of traffic control for the reserved-lane operation were considered. It was determined that a relatively low-cost, fixed-message traffic signing plan was the most feasible for the Shoreway. Variable-message signing and blankout-message signing were rejected, mainly because of the high initial capital investment and the high annual operating cost required.

USER COSTS

A detailed user-cost analysis was performed to compare the reserved-lane concept with normal freeway operation. All direct costs incurred by all users of the freeway during the commuting hours were accounted for in the analysis. For those who use the freeway, all costs (both on and off the freeway) were included. The following cost elements were considered:

Element	Cents
Direct costs of vehicle operation per vehicle-mile	5
Parking costs per vehicle	58
User time costs per person-hour	90
Bus fare per trip	38

The approach used in the evaluation was to calculate differences in operating costs resulting from changes in travel behavior and changes in freeway operating characteristics brought about by implementation of the reserved-lane concept of operation. Cost differences were calculated for each hypothetical shift into car pools and buses that was studied with the freeway-flow model.

Figure 6. Schematic diagram of queuing under reserved-lane operation.

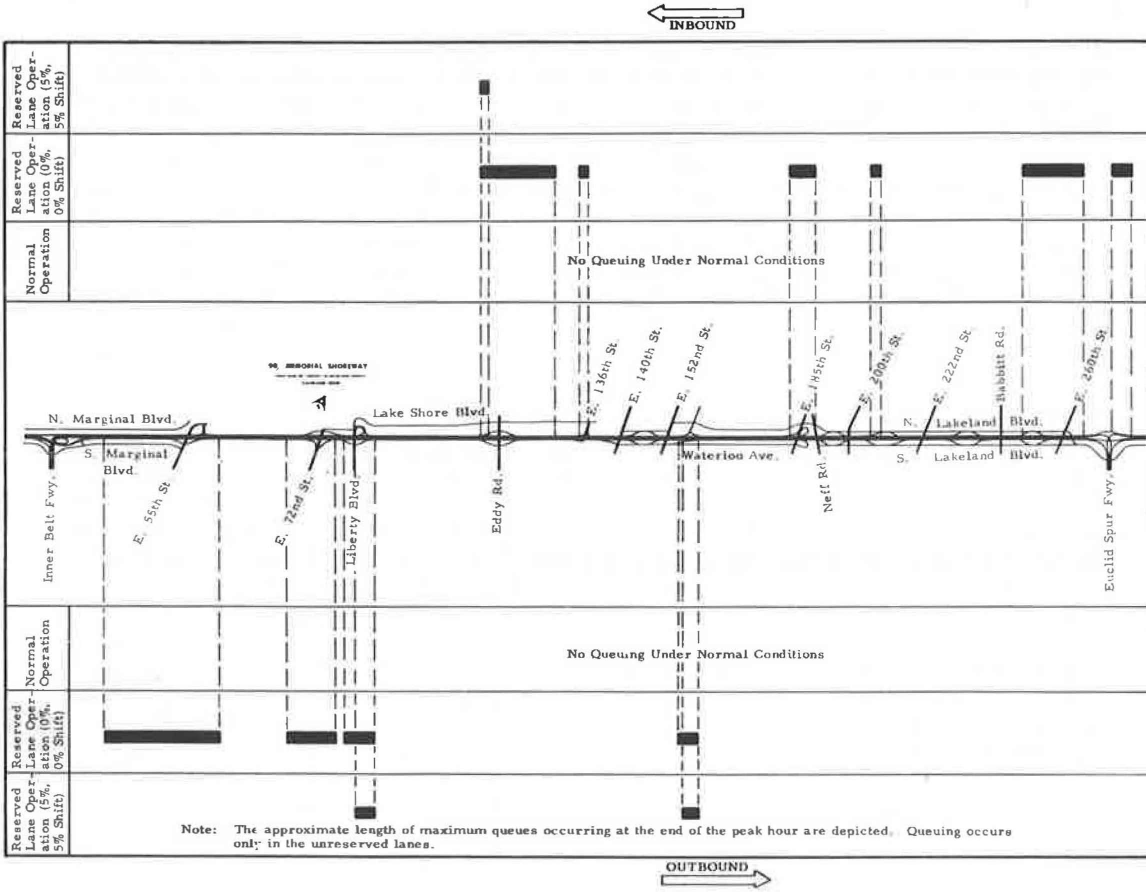
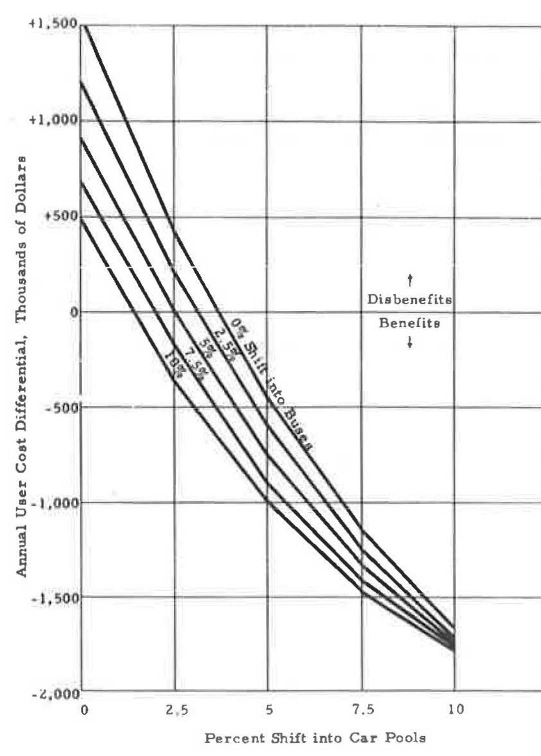


Figure 7. Annual user cost differential resulting from reserved-lane operation inbound from 6:30 to 9:30 a.m. and outbound from 3:30 to 6:30 p.m.



It was found that, initially, reserved-lane operation results in substantial disbenefits, attributable entirely to increased user-time costs on the freeway. As shifting occurs into car pools and buses, disbenefits are progressively changed to increasing benefits. The greater the degree of shifting is, the greater the benefits are. Figure 7 shows annual user benefits for whatever degree of shifting is predicted or ultimately occurs. For example, if the reserved-lane concept induces a 5 percent shift into car pools and a 5 percent shift into buses, then approximately $750,000 in annual user benefits are obtained. A lesser degree of mode shifting is required to yield monetary benefits than to reduce total hours of passenger travel as can be seen from a comparison of curves shown in Figures 2 and 7.

In addition to the quantifiable benefits, there are other potential benefits that do not lend themselves to quantification. They include improved reliability of travel time for bus and car-pool travelers, increased bus service and efficiency, reduced need to widen freeways, reduced air pollution, reduced need to increase the number of automobiles owned by households, reduced need to use the central business district land as a "day care center" for the automobile, and increased use of more efficient forms of moving people by the "prestige value" associated with giving them preferential treatment.

MODAL CHOICE

The degree to which the various potential benefits of the reserved-lane concept will induce drivers to switch to car pools or to buses was analyzed by means of a "marginal utility" model, which uses travel time and out-of-pocket costs as major measures of the attractiveness of each mode of freeway transportation. The general form of the utility (or disutility) function was $U = F(T, C)$. The exact form of the disutility functions for automobile drivers U_a, automobile passengers U_p, and transit passengers U_t was as follows:

$$U_a = 2.5\,A_t + A_r + (0.5P + 5D)/C$$
$$U_p = 2.5\,A_t + A_r + (0.5P + 5D)/(C \times L) + (L - 1) \times E$$
$$U_t = 2.5\,(T_a + T_w) + T_r + F/C$$

where

A_t = automobile terminal time;
A_r = automobile running time;
P = parking cost, cents;
D = highway distance, miles ($5D$ = vehicle operating cost, cents);
T_a = walk time to and from transit;
L = occupancy level;
E = penalty per extra passenger, min;
T_w = wait time for transit;
F = transit fare; and
C = cost of time (computed as 25 percent of mean income of trip-maker), cents/min.

The model was calibrated to existing conditions, and several iterations were performed for various changes in freeway operating speeds.

For the maximum condition, person trips on the freeway dropped by 458, transit trips increased by 523, and automobile occupant trips decreased by 981. These corresponded to about a 3 percent shift into buses and a 3 percent shift into car pools.

The results must be interpreted with care because of the limited nature of the model and the data base used to exercise it. Furthermore, the model did not account for the potential efficacy of an aggressive public information program. Also, the model was not used to analyze increased ridership that could be obtained through increased bus service on existing routes and establishment of service on new routes in the corridor. It is considered likely that, with such programs, the actual shifts in travel behavior will substantially exceed those predicted by the model, and shifts greater than 5 percent into buses and 5 percent into car pools would not be unrealistic.

LEGISLATION AND ENFORCEMENT

Legislation

A review of existing legislation in Ohio was conducted to determine whether new legislation would be needed to eliminate any possible conflicts with the proposed reserved-lane concept. It was found that enactment of some new local ordinances would be sufficient and that such legislation is feasible.

Attitudes of Police and Courts

Police officials and judges from the areas affected by the freeway section under study were interviewed to determine their reactions to the reserved-lane concept. In general, each of the agencies was skeptical of the concept's practicality. They believe that enforcement will be extremely difficult and that the public will resent the plan. It is clear that a positive information program must be developed and presented to these officials prior to concept implementation, for without their full support an effective enforcement program will be difficult to achieve.

Enforcement

Obedience to lane-use restrictions was secured by establishing that 2 conditions must prevail: Every driver must clearly understand whether he may or may not use the reserved lane at any given time, and there must be an effective deterrent to encroachment on the reserved lane at the wrong time.

It was concluded that the usual traffic enforcement tactics of apprehension and citation would not be well suited for use under the reserved-lane concept of operation. In other words, greater reliance will have to be placed on voluntary compliance (through an effective public information program) and deterrence (through a program of increased concentration of police patrols).

PUBLIC RELATIONS, INFORMATION, AND EDUCATION

The success of a demonstration of the reserved-freeway-lane concept depends on how well a public relations and information-education program is implemented. Those involved in and affected by the demonstration must be made aware of its importance and the anticipated benefits and advantages, thoroughly briefed on the operation aspects, and persuaded that their cooperation serves their own interests as well as those of the community.

It became evident from interviews with citizens and public groups in Cleveland that acceptance of the reserved-lane concept will require a comprehensive public relations and information-education program. The idea must be promoted to the public in a positive manner to offset the negative attitude likely to be created by restricting highway movement.

From a public education-public relations viewpoint, the concept is viable—provided it is technically feasible. It is possible to create and implement a program that will result in a high degree of cooperation by officials, users, and potential users of the facility. The criteria for such a program must identify the target audiences to which the effort must be aimed and forcefully bring to their attention the merits of the project; stimulate and sustain interest in the concept among the key audiences; create a desire among the target audiences to participate in the project and to support it; and motivate the majority to act.

CONCLUSIONS OF FEASIBILITY STUDY

General

1. The concept of reserving an urban freeway lane for the exclusive use of buses and car-pool vehicles during commuting hours is basically sound.

2. Reserved-lane operation will result in a reduction in the total number of vehicles required to serve a given level of travel demand when sufficient numbers of commuters shift from low- to high-occupancy vehicles and from low-occupancy cars into buses.

3. Feasibility of the reserved-lane concept depends largely on the characteristics of each specific freeway. Freeways with 4 or 5 lanes in each direction provide the greatest amount of flexibility.

4. No dramatic changes in highway safety can be expected with the introduction of reserved-lane operations. It is believed that many of the important influencing factors will tend to nullify each other.

Memorial Shoreway

5. The reserved-lane concept on the Memorial Shoreway in Cleveland, Ohio, is technically feasible and warrants a full-scale demonstration.

6. Implementation of the reserved-lane concept will produce the desired effects on traffic operation, as indicated by the detailed freeway-flow analysis.

7. If the reserved-lane concept works under ideal conditions (free of capacity-impairing incidents), it will also produce operational improvements on days when accidents occur.

8. The reserved-lane concept can accommodate future travel growth but should be introduced while traffic demands are still moderate rather than after congestion has become severe.

9. The left lane should be used as the reserved lane.

10. Given the anticipated degree of shifting, traffic demand-capacity relations in the unreserved lanes will approximate existing normal conditions. Only a few minor, localized points of congestion will remain.

11. Distances in the range of 5,400 to 7,500 feet are required for smooth transition from normal to reserved-lane operation at the beginning and end points of the selected freeway section. The design of the traffic control plan provides for transition sections of the required length.

12. Shifts in mode are necessary to increase the freeway's person-carrying capacity. The reserved-lane scheme developed for I-90 has the potential of increasing this capacity by approximately 13 percent.

13. No physical changes are required to accommodate reserved-lane operation.

14. Reductions in user cost are a function of the degree of shifting from low- to high-occupancy vehicles.

15. Side benefits of reserved-lane operation include improved travel-time reliability, diverted short freeway trips, improved bus service and efficiency, deferred costs of future facilities, reduced air pollution, decreased automobile ownership costs, converted central district parking land, and increased prestige for public transportation.

16. Legislation required for the implementation of reserved-lane operation appears feasible.

17. Because apprehension tactics are not well suited for use with reserved-lane operation, emphasis must be placed on voluntary compliance and on increased concentration of police patrols.

18. An aggressive and effective public information program is vital to the success of the demonstration of the reserved-lane concept.

CLOSING REMARKS

It was discouraging to learn that the Cleveland area agencies that would have to take action to implement the recommendations of this feasibility study decided not to do so. We are hopeful that a demonstration implementation will be undertaken at some other site, however. Plans have been implemented to lengthen the exclusive bus lane to the toll plaza on the Oakland Bay Bridge to include 2 lanes reserved for vehicles with 3 or more occupants. Although this is a step in the right direction, it does not meet the need for a demonstration as recommended by the feasibility study because, with the exclusive lane traffic passing through the tollbooths on the bridge, there will be a built-in enforcement mechanism. The real test is whether the concept will work and can be enforced under the typical freeway conditions found commonly across the country. At the time this paper was written, the Florida Department of Transportation was giving this concept serious consideration for a facility in the Miami area.

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INVESTIGATION OF LANE DROPS

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Objectives of the study were to evaluate the effectiveness of safety and traffic operations of lane-drop configurations on several freeways. Aerial photographs were taken of traffic volumes at 3 lane-drop sites in the Los Angeles area, time-history trajectories of all vehicles were computed, and measures of traffic operations (speeds and concentrations) and safety (potential collisions) were determined. No significant differences in safety effectiveness among the 3 configurations were identified, but 1 site at which the pavement markings caused the 2 heavily traveled left lanes to merge had significantly lower speeds and higher concentrations than either of the other sites. The aerial photographic data reduction system developed during the study allows objective analysis of the complex interaction of traffic at lane drops or at other freeway configurations. However, the system needs further development before it can be an economical tool for the traffic engineer.

•IN 1971, the System Development Corporation completed an 18-month study to evaluate the effectiveness of several lane-drop design configurations with respect to safety and traffic operations. This paper summarizes that project and reviews the study findings.

PURPOSE AND TECHNICAL APPROACH

It is sometimes necessary to reduce the number of lanes on a freeway because of funding limitations or because the expected traffic demand is decreased outside of metropolitan areas. Because many variables affect the operating conditions and safety of the various lane-drop configurations, sound criteria for the selection of the proper lane-drop design for various traffic and freeway geometric conditions are needed. The objectives of this study were to

1. Determine from field data the effectiveness of existing main-line lane drops from the standpoint of safety and traffic operations;
2. Determine the effects of the significant parameters associated with various levels of safety and traffic services; and
3. Recommend configurations for lane drops based on the findings of objectives 1 and 2.

The technical approach to the study consisted of the following tasks:

1. Obtain information about the locations, configurations, traffic conditions, and accident experience at all lane-drop sites on the California freeway system and select several sites for detailed study;
2. Modify an aerial data reduction system to produce trajectories (time histories) of the distance, lane position, and velocity of all vehicles traveling through the freeway segment containing a lane drop during observation periods up to 15 min long;
3. Design and program measures of traffic operations and safety effectiveness that could be used in the analysis of trajectories produced by the data reduction system;
4. Conduct a pilot study by collecting, reducing, and analyzing aerial film from 1 or 2 sites to check out the measures and the data reduction system;

Sponsored by Committee on Freeway Operations.

5. Film and analyze additional sites with various configurations to evaluate traffic service and to determine the effects of variations in significant parameters on service; and
6. Develop design recommendations for lane-drop configurations and locations.

Because only 3 sites were filmed and analyzed, it was not possible to complete extensive parametric evaluation or to develop more than preliminary design recommendations. Additional studies are planned to complete attainment of the project objectives.

DATA ACQUISITION AND REDUCTION

Site Selection

It was initially assumed that 7 sites would be studied, encompassing at least 5 different lane-drop configurations. It was planned to collect data at 3 different traffic volume levels: fewer than 1,000 vehicles/lane/hour; 1,000 to 1,500 vehicles/lane/hour; and more than 1,500 vehicles/lane/hour. The sites were to be selected from a list of nearly 200 lane-drop locations compiled by the California Division of Highways.

An examination of aerial photographs and traffic counts of many locations showed that fewer than half of the lane drops were of the 5 configurations desired and that few had peak traffic counts of more than 1,000 vehicles/lane/hour. Sites selected for the pilot tests were where the right lane dropped at an off-ramp and counts during afternoon peaks were more than 2,000, where the right lane tapered off and Sunday afternoon peaks were about 1,000, and where the right lane dropped and striping merged the 2 left lanes. A fourth site that had a trapped lane off-ramp and high afternoon counts was filmed, but the analysis was not completed.

Data Collection

Before aerial filming could begin, it was necessary to survey the sites and install reference markings. Traffic counts were conducted at each site on several days so that a reasonably accurate estimate of flow levels during different time periods could be made.

The camera used for data collection was the Maurer 220 pulse-sequence camera, designed for aerial reconnaissance. The camera was mounted on a helicopter. A 38-mm Zeiss Biogon wide-angle lens with a relative aperture of f/4.5 was used, allowing filming about 1 mile of freeway from the helicopter hovering at 4,000 ft. Each film magazine held somewhat more than 100 ft of 70-mm Kodak Ektachrome MS aerographic color film, allowing filming about 8 min of traffic when a 1-sec frame interval was used (a 2-sec interval was also used during relatively light traffic flows). Each site was filmed for at least 3 different intervals; because of some camera jamming, 1 site was filmed for 6 shorter intervals, one of which was discarded because of extensive traffic-flow breakdown.

Data Reduction

The film was read on a Bensen-Lehner model 29E telereader. Each frame was projected on a 28- by 28-in. matte reading surface, and the operator located cross-hairs over each reference point and vehicle position. The film-reader coordinates and reference information such as film number, dates, and locations were punched on cards. After the cards were edited and stored on tape, they were processed through a program that corrects the photographic image points for the optical distortion produced by the combined effect of the camera lens and film magazine, determines the position and orientation of the aerial camera in ground coordinates, and computes the corrected car positions in the chosen ground coordinates. [The space resectioning methods follow Keller and Tewinkel's work (1).]

The resection outputs were processed by a trajectory program in which the geometric characteristics of the site are described, all vehicles are located relative to the boundaries of the freeway test section, and a given vehicle's positions from frame to frame are matched to produce a trajectory of the lane location, distance along the test section, and velocity.

The film-reading and data reduction procedures are described in detail in another report (2).

ANALYSIS OF TRAFFIC OPERATIONS

Analytical Techniques

The traffic-operations measure most commonly used is average travel time (or alternatively average overall speed). Although this type of measure gives a good indication of the general nature of the flow of traffic through a test section, it was not found to be an adequate measure for comparison of lane-drop configurations. Differences in desired speeds and in the geometry of the sites (other than those attributable to different lane-drop configurations) made the comparison of average overall speeds or travel time for different sites of little value. Those differences tended to obscure the effects of the lane drop itself on speeds and travel times.

To compare the effects of different lane-drop configurations on traffic required the development of an analytical description of the changes in speeds and in flow patterns throughout the lane-drop area. A photographed site was partitioned into short subsections, and local estimates of flow and space mean speed for each lane were calculated at the boundary points of each subsection. The flow at a boundary point was given by the number of vehicles passing the point during the filmed period, adjusted to yield cars per hour. The space mean speed at the boundary point for each lane was calculated as the harmonic mean of the speeds of vehicles observed passing the boundary point.

The values of speed and flow for each lane at each boundary point were then plotted against the distance upstream from the end of each site.

This procedure gives a very promising analytic tool for tracing the effects of lane-drop geometry on traffic operations. The graphs of speed and flows as functions of distance upstream from the exit point of each site are given in the following section. Looking at the speed graphs in particular, we can trace the effects of the lane drop at various volumes. Also, we can distinguish which disturbances were due to the lane drop and which were due to other nearby geometric features.

Furthermore, a statistical analysis indicates that, even for the short time periods of data that we have, the estimates of space mean speed have very small errors. The order of magnitude of the errors relative to the speed itself is about ±1 percent. Therefore, the random deviation of the estimated speed curve from the true mean speed curve is small. This assures us that the analysis of the mean speed curves will lead to statistically reliable results.

The 3 sites for which film was collected and reduced were all located in the greater Los Angeles area. Figures 1, 2, and 3 show diagrams of the 3 sites. Project personnel drove through all of the sites many times and observed traffic operations from nearby overpasses.

Site 1 Description

Site 1 is on the Ventura Freeway inbound between the Calabasas Parkway and Mulholland Highway overcrossings. Both the pavement and the lane markings are such that the right lane is dropped. The lane drop occurred as a result of stage construction and is a transition from a newer stretch of 4-lane concrete (opened in 1967) to an older section of 3-lane asphalt. The 3-lane section continues for only about 2 miles before increasing to 4 lanes. The lane-drop taper starts at milepost 27.76, continues through a 2-deg right curve in the freeway, and ends at milepost 27.57. The lane drop is preceded by a low-volume on-ramp from Calabasas at milepost 27.90 and is followed by a low-volume off-ramp to Mulholland Highway at milepost 27.36. The only sign indicating the lane drop is placed between the on-ramp and the start of the lane-drop taper, but the sign and striping are clearly visible at least a mile upstream to drivers approaching on a long 1.5 percent downgrade. About 5 miles upstream of the lane drop the freeway has only 2 lanes, and there is only 1 on-ramp between the end of the 2-lane section and the ramp immediately preceding the lane drop, so that traffic flow is somewhat limited. The freeway climbs a hill for about 3 miles, and then gradually descends until it levels off about at the lane drop. Most of the automobile traffic appears to move to the left 2 lanes to avoid trucks slowed by the hill, and only about 10 percent of the vehicles remain in the right lane at the start of the test section ½ mile upstream of the lane-drop taper.

Figure 1. Plan and elevation view of site 1 on Ventura Freeway at Calabasas.

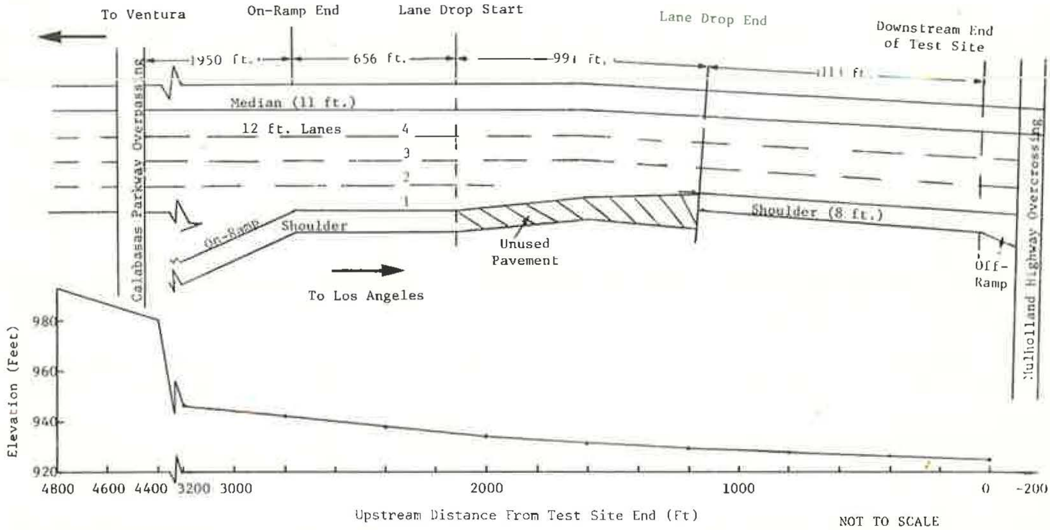


Figure 2. Plan and elevation view of site 2 on San Bernardino Freeway at Holt Avenue.

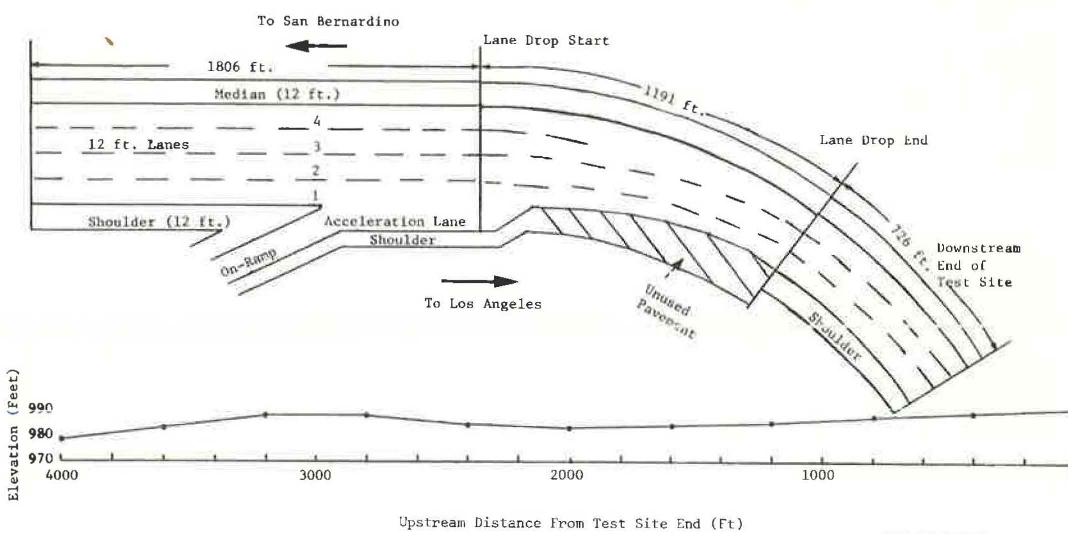
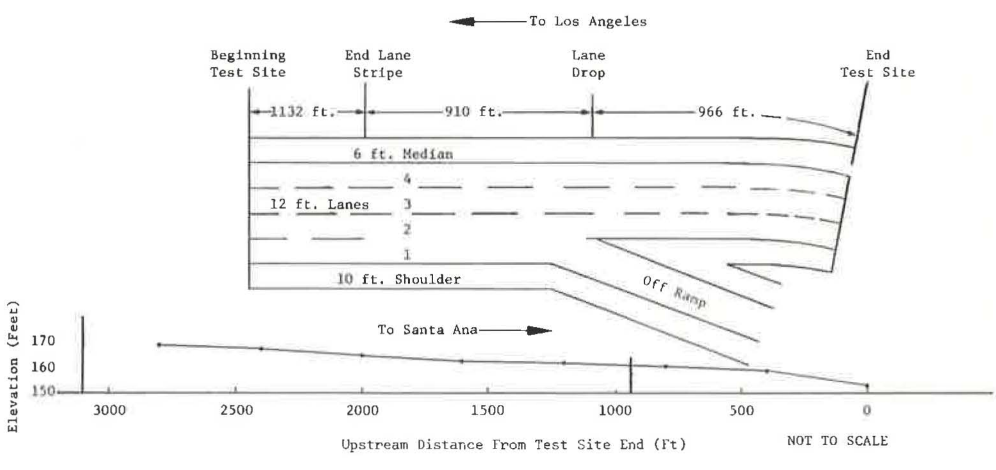


Figure 3. Plan and elevation view of site 3 on Santa Ana Freeway at Triggs Street.



The heaviest traffic flow at the site occurs on Sunday afternoon and consists mainly of automobiles returning to Los Angeles from recreational areas. Less than 5 percent of the vehicles are trucks. Traffic increases throughout Sunday afternoon, and the peak occurs in the early evening. Speeds on the section are generally higher than the 65-mph limit, probably because of the long, smooth downgrade and good visibility, but the end of the slope and the rougher pavement encountered after the lane drop seem to slow all vehicles.

Site 2 Description

Site 2 is comparable to site 1 in pavement configuration and traffic characteristics except that the pavement striping merges lane 3 into the left lane and steps the 2 right lanes to the left. Site 2 is on the San Bernardino Freeway inbound between the Archibald Avenue and Vineyard Avenue overcrossings. At the time the site was filmed, the pavement ended on the right; since then a fourth lane and an off-ramp on Vineyard Avenue have been added on the right. The lane-drop taper, as indicated by the striping on the freeway shoulder, starts at milepost 6.65 and continues to milepost 6.43, but the sign indicating that the left 2 lanes should merge is at milepost 6.90. The sign indicates that lane 4 should merge right, but actually lane 4 continues to follow the arc of a 6,500-ft radius, right curve in the freeway, and the third lane is merged left into lane 4. The Holt Avenue on-ramp enters the right lane at milepost 6.80 but carries very little traffic.

Heaviest traffic at this site is also on Sunday afternoon and evening and consists of vehicles returning from weekend recreation. Traffic flow occasionally breaks down in the early evening hours because of reduced capacity downstream. Between 5 and 10 percent of all vehicles are trucks. Traffic was observed to flow in marked platoons, more so than at site 1, but the effects of the lane-drop merger on heavy platoons of traffic in the 2 left lanes slowed many vehicles considerably. This phenomenon will be discussed below in greater detail.

Site 3 Description

Site 3 has the right lane trapped onto an off-ramp. It is located on the outbound Santa Ana Freeway at the Triggs Street off-ramp, milepost 13.17. The 4-lane section starts at the Long Beach Freeway interchange, about 1 mile upstream of the lane drop, but traffic from the Long Beach Freeway enters the Santa Ana Freeway from a left ramp. Immediately downstream of the lane drop, the freeway curves to the right and descends to pass under 3 overcrossings; the fourth lane reappears briefly as an auxiliary lane between the Atlantic Boulevard South on-ramp (near milepost 12.95) and the Atlantic Boulevard North off-ramp (milepost 12.75). There is also an on-ramp from Triggs Street at milepost 13.04 and an off-ramp to Atlantic Boulevard South at milepost 13.00.

Traffic at this site is very heavy every weekday afternoon and usually reaches breakdown conditions between 3 and 4 o'clock. The breakdown appears to be caused by a backup into the lane-drop area from downstream rather than by the lane drop. More than 20 percent of the traffic consists of trucks (although film was collected and reduced for 1 period during the Teamster Union strike when less than 5 percent of traffic was trucks; the lane drop is located in an industrial and commercial area. The late afternoon traffic is mostly commuter traffic from downtown Los Angeles to the residential areas southeast of the city.

Site 1 Traffic Operations

The results for site 1 show a uniformity of behavior at all flow levels (Figs. 4 and 5). The speeds in each lane remain approximately constant until two-thirds of the way through the lane-drop taper. At this point speeds in all lanes decrease linearly until the end of the test section. The rate of decrease in all lanes is approximately the same. The uniformity in speed decrease in all lanes suggests that the cause of decrease is not the lane drop itself but rather some other geometric feature. In fact, the site description shows that at the point where speeds begin to decrease a long downgrade (which extends upstream of the beginning of the test section) flattens out and driver sight distance

Figure 4. Traffic flow rate at site 1 on June 7, 1970.

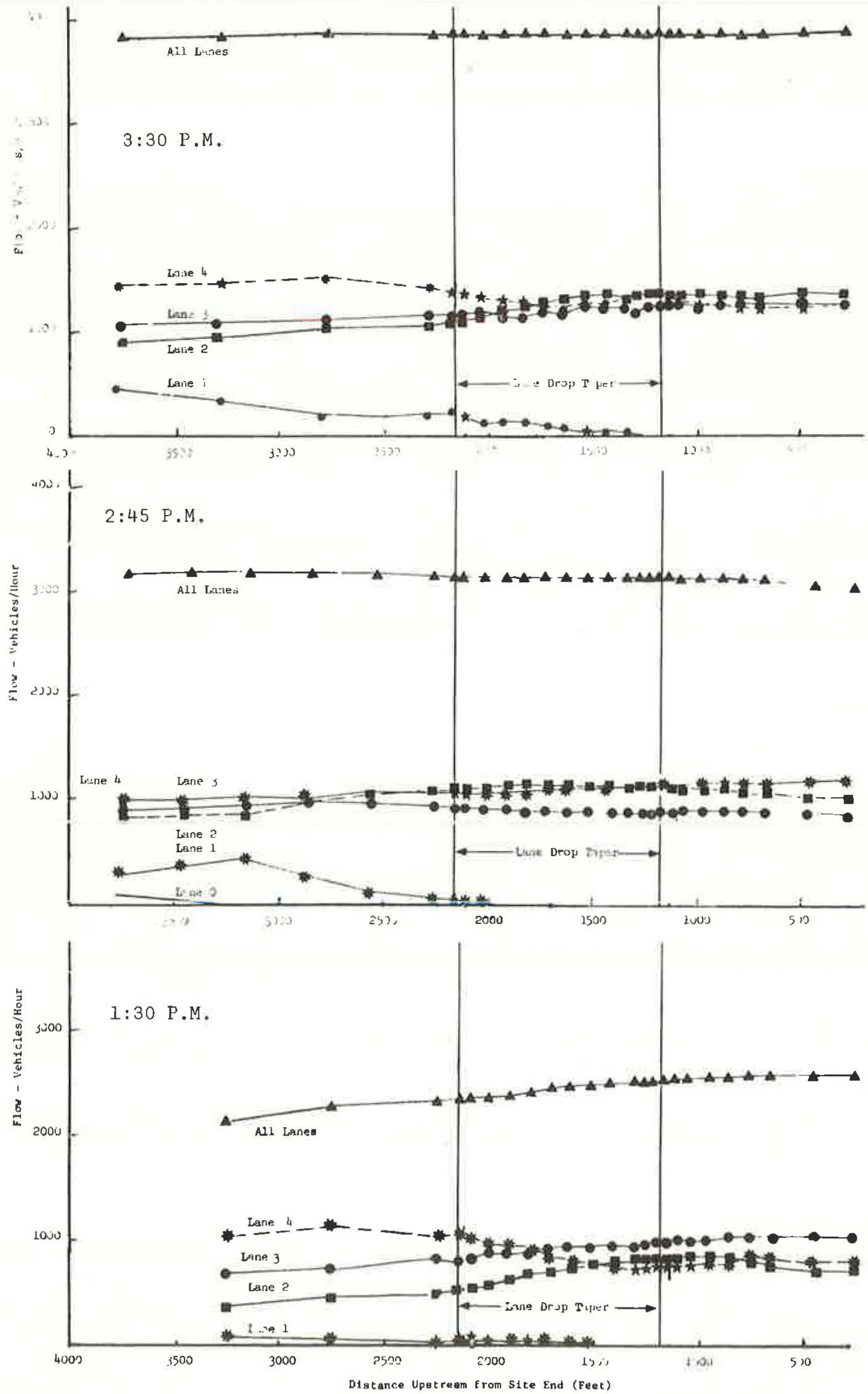
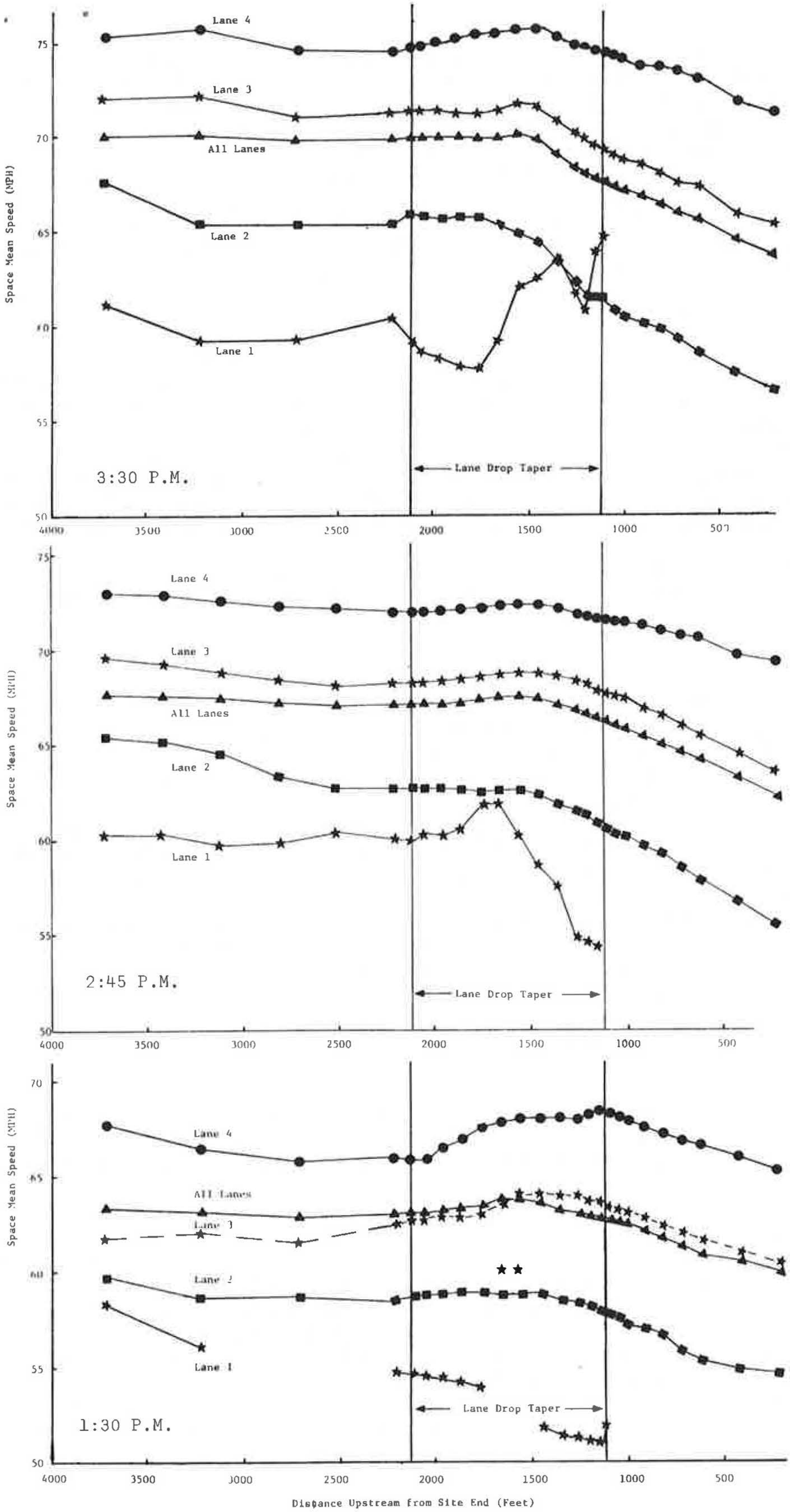


Figure 5. Space mean speed at site 1 on June 7, 1970.



is decreased; also, there is a transition from smooth concrete to rougher asphalt. These features probably account for much of the change in speeds. The high speeds observed at the beginning of this site are partially due to the long downgrade but are not atypical for California freeways. Similar speeds at the same site have been confirmed by many observers on several Sunday afternoons.

Figure 4 shows that the flow in lane 1 at the beginning of the test section is already less than 10 percent of the total flow. Light flows in the right lane are characteristic of all sites studied. The movement of vehicles from lane 1 to lane 2 is smooth and gradual, and almost no cars are in lane 1 near the end of the taper; the fluctuations of the speed curve in that region are not considered significant.

Site 2 Traffic Operations

The results for site 2, in which lane 3 is merged into lane 4, also show a uniformity of behavior at all flow levels (Figs. 6 and 7). The 2 lanes that merge at the lane drop show a severe decrease in speeds; however, lane 1 has almost no speed drop and lane 2 has only a slight drop. The disturbances in speed in all lanes increase as the flow increases. The traffic flow rate curves indicate the vehicles in lane 4 follow the signing and start to merge right upstream of the lane drop, then are forced back to the left by the pavement markings in the lane-drop taper area. Flows in lanes 1 and 2 remain relatively undisturbed except for the influx of a few vehicles from the on-ramp upstream of the lane drop.

The slight speed increase in lanes 1 and 2 at the upstream end of the lane-drop taper was observed during all 3 film periods, but no explanation of its cause has been found.

Site 3 Traffic Operations

The speeds at site 3 show a gradually changing pattern of speed decreases as flow increases (Figs. 8 and 9). At the moderate flow levels (the lightest flows filmed at this site were commensurate with the highest flows filmed at sites 1 and 2), speeds in lanes 2 and 3 slightly decreased in the lane-drop area and recovered in the downstream area. At this flow level there was no disturbance of speeds in lane 4.

As flow increased, the decrease in speeds in lane 2 became intensified, and the recovery in lanes 2 and 3 was delayed. Only at the highest flow level analyzed was there even a slight decrease in the speeds in lane 4. At this flow level no recovery of speed was seen in lanes 2 and 3.

Figure 8 shows that only about 10 percent of the flow was carried by lane 1 at the beginning of the test section. Most of those vehicles accomplished their merge into lane 2 well before the end of lane 1, but a small proportion of cars made their lane changes in the last 50 ft before the off-ramp.

ANALYSIS OF TRAFFIC SAFETY

Accident Analysis

It was originally proposed that the traffic safety of different lane-drop site configurations be evaluated by using accident data collected at various sites. Because it was expected that the sensitivity of inferences drawn from accident data would be limited by the necessity of using long time averages in order to obtain stable accident statistics and by the grossness of the traffic volume data available, this approach was to be augmeted by the analysis of aerial data.

The results obtained by Tye (3) confirmed our expectations about the limited usefulness of accident data. In this study the relative safety of 147 lane-drop sites on California freeways was explored through the use of 1965 and 1966 accident report records. In the study, no definitive results were obtained indicating any differences in the accident experience at the different lane-drop configurations considered in the present study. The variation of observed accident rates was greater within each configuration than among configurations. Tye indicates that the failure to discover significant differences among configurations may result from factors such as inaccurate

Figure 6. Traffic flow rate at site 2 on July 26, 1970.

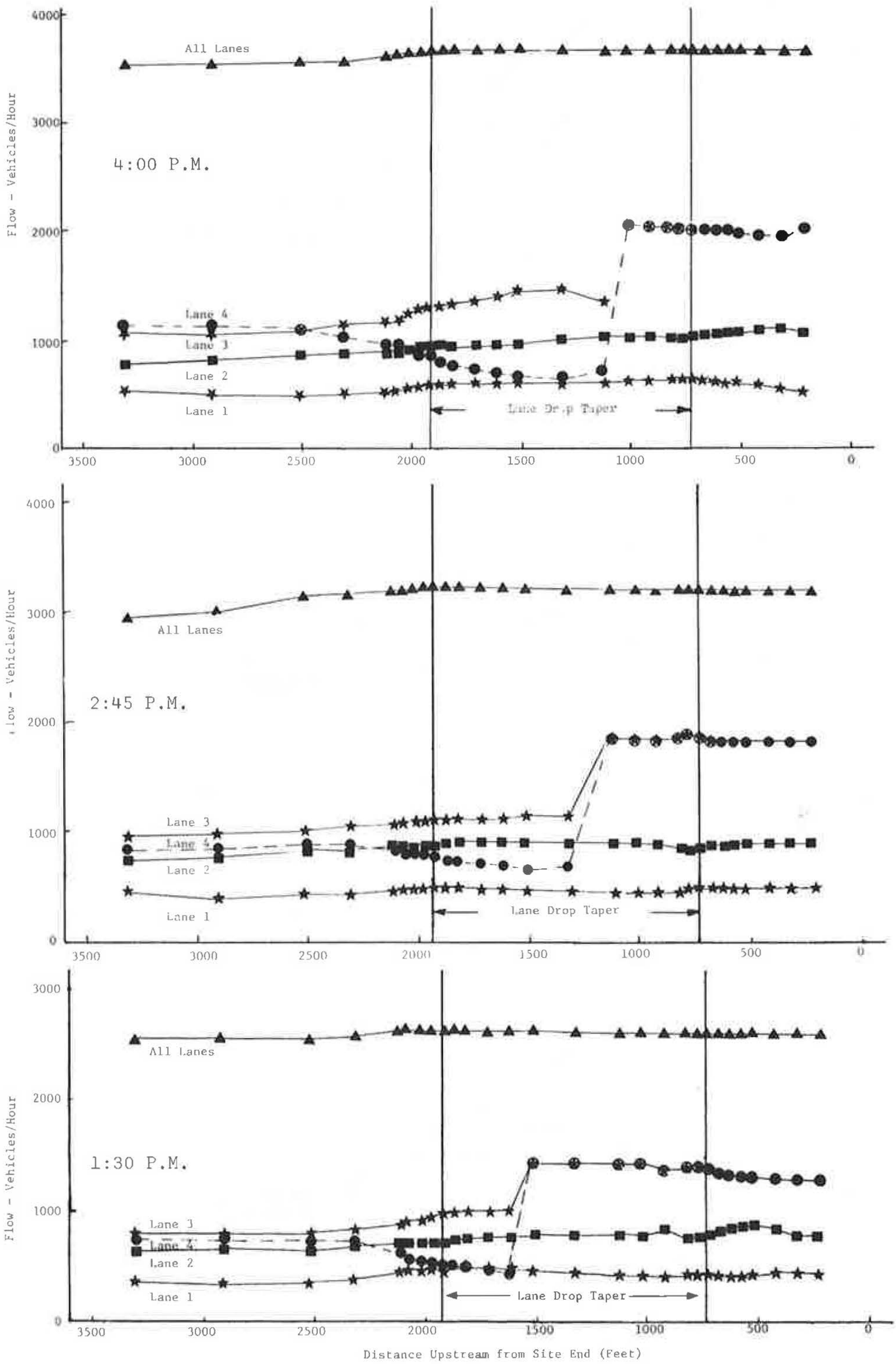


Figure 7. Space mean speed at site 2 on July 26, 1970.

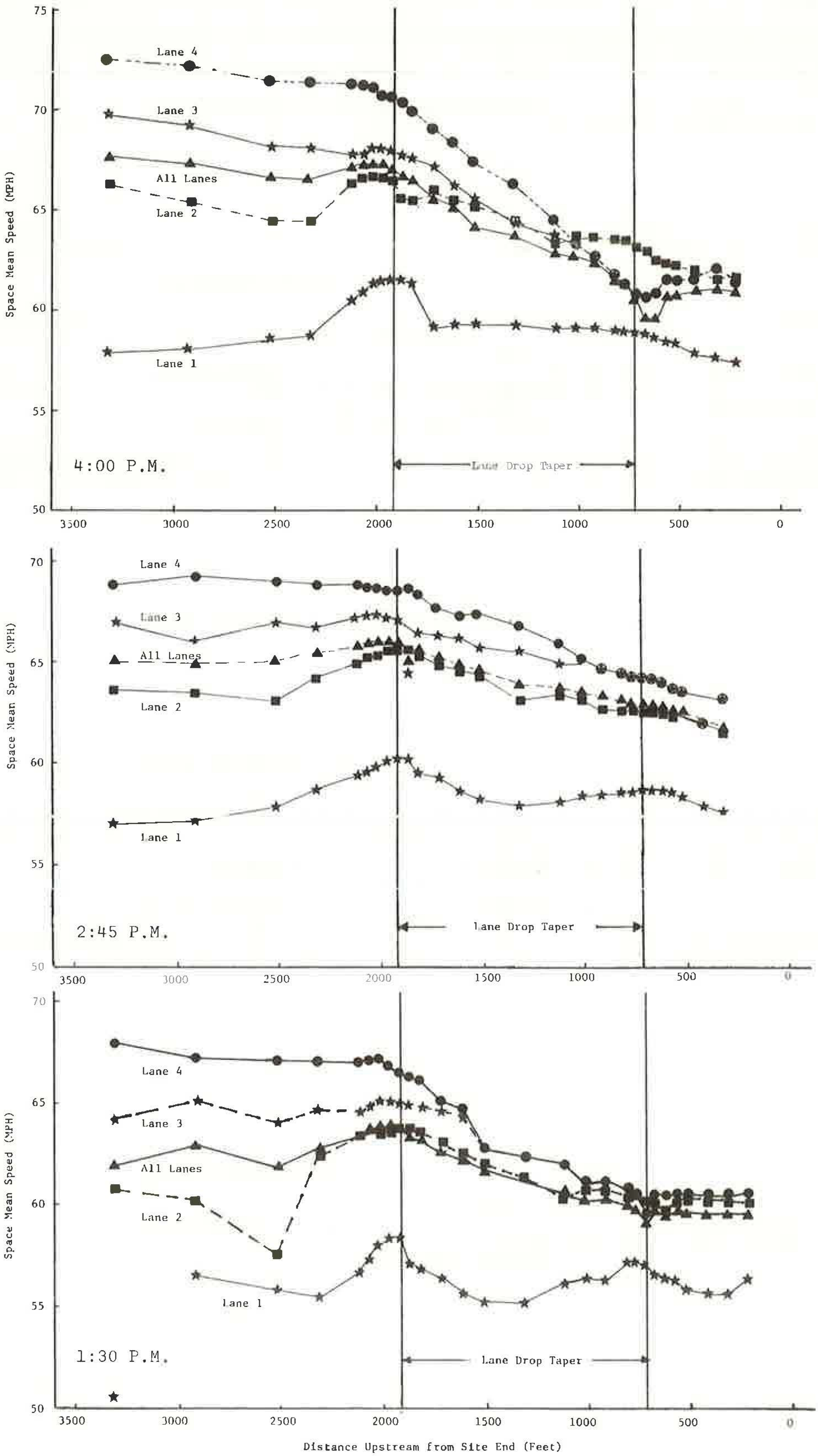


Figure 8. Traffic flow rate at site 3 on May 6 and June 3, 1970.

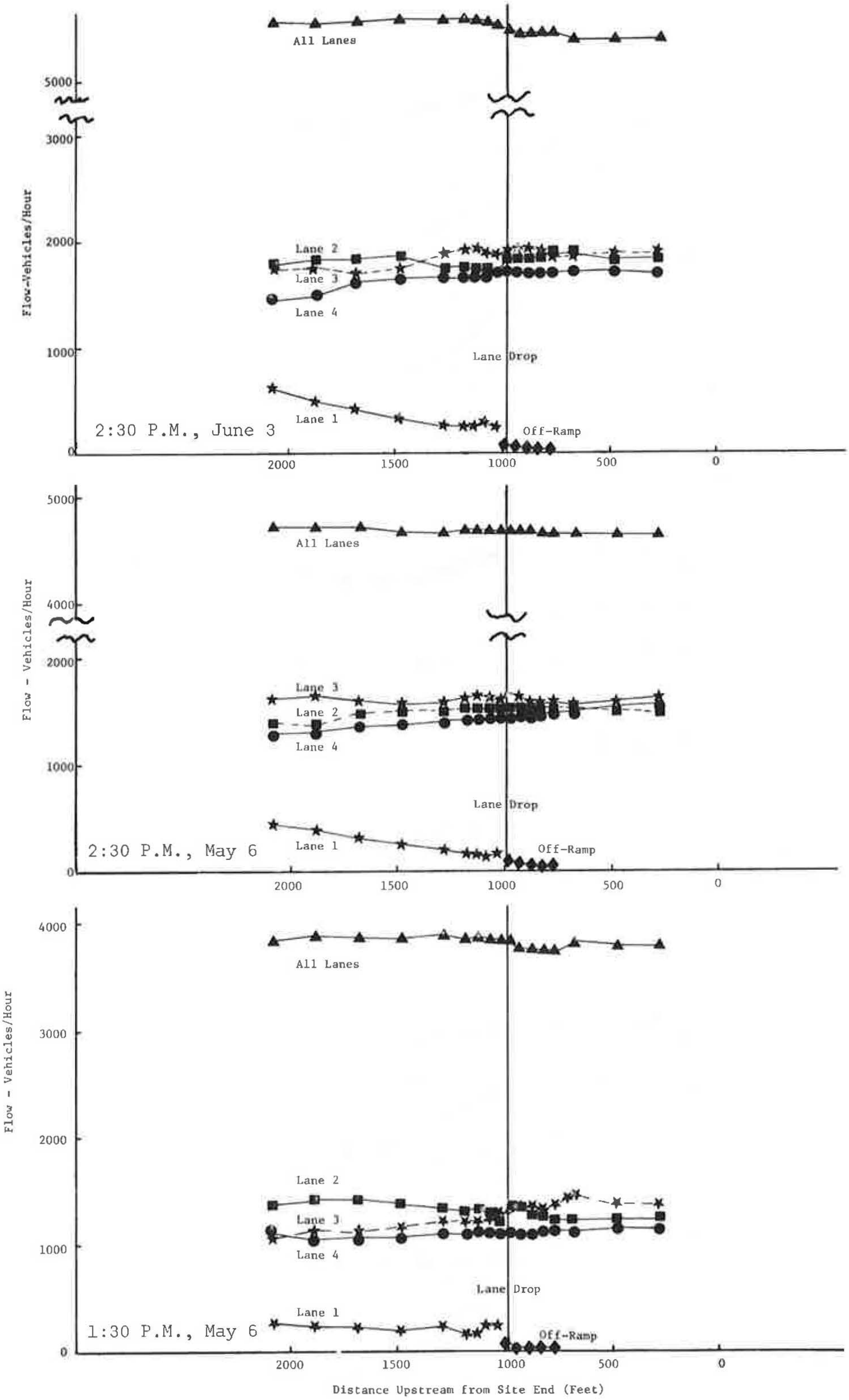
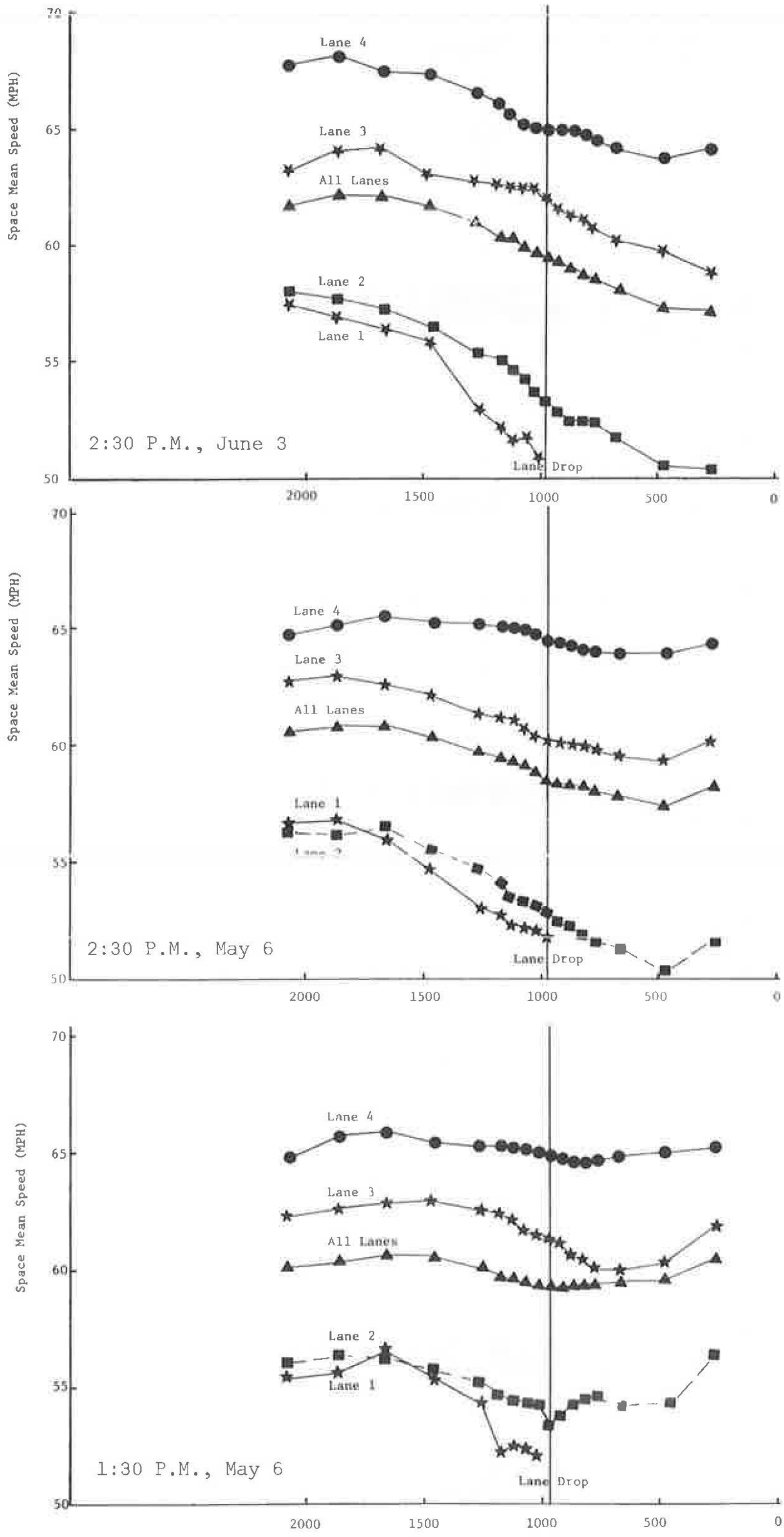


Figure 9. Space mean speed at site 3 on May 6 and June 3, 1970.



traffic volume information, masking of the effect of a lane drop by the presence of some other conflict-producing geometric feature, or variation in accident-reporting procedures from location to location or agency to agency.

Despite the problems in developing conclusive results encountered by Tye in his study, accident report summaries for the sites under study were obtained from the California Division of Highways, and a brief analysis was conducted.

At site 1, only 1 of the 7 accidents reported in the 2-year period seems lane-drop related. Most accidents appear to result from vehicles traveling too fast down the hill preceding the lane drop and running off the road into the median fence. At site 2, 4 of 19 accidents reported seem lane-drop related, and several others occurred during the heavy congestion on Sunday evening. The accident rate (for similar annual travel volumes) was significantly higher at site 2.

At site 3, there were at least 11 of 40 accidents reported of a type that might be lane-drop related. There were many more accidents reported at this site, in a somewhat shorter section than either of the other two, but the annual traffic volume at the site was about 3 times that at the others. Most of the accidents at site 3 involved vehicles stopping during the afternoon congestion.

However, the number of accidents reported in the immediate vicinity of the lane drops is no greater than the number reported in adjoining areas. Figure 10 shows the number of accidents reported in ⅕-mile intervals near the lane-drop sites. These graphs indicate that the lane drops investigated do not have higher accident rates than the nearby sections of 3- and 4-lane freeways.

Analytic Techniques

The traffic-safety measures of a lane-drop site are defined for the field experimental program in terms of "hazardous conditions." If one observes a set of trajectories (determined by aerial photography techniques) along a lane-drop site where vehicle 1 has speed v_1 and vehicle 2 has speed v_2 and both vehicles are in the same lane, then the hazard associated with the 2 vehicles will depend on the separation distance, $\Delta x = x_1 - x_2$, and on the speeds of the 2 vehicles, v_1, v_2. It is hypothesized that there is a region defined by a function of v_1, v_2, and Δx, such that when $(v_1, v_2, \Delta x)$ is in this region the probability of a collision is substantially greater for the corresponding pair of vehicles.

The formulations described below for determining the occurrence of hazardous conditions were applied to data collected by using the aerial photography system. After trajectories have been produced, at each of the intervals $i\Delta t$ $(i = 1, 2, \ldots, N)$ all pairs of car-following vehicles that fall within the lane-drop test section are determined. Each car-following pair is then tested to determine whether it falls in the hazardous region. For each $i\Delta t$, there result an h_i, which is the total number of car-following pairs that fall within the hazardous region, and an N_i, which is the total number of car-following pairs at $i\Delta t$. The response, or safety-effectiveness measure corresponding to a lane-drop site configuration, is then given by

$$R = \sum_{i=1}^{N} h_i \Bigg/ \sum_{i=1}^{N} N_i$$

where N is the total number of observation instants. R is termed the "hazard ratio" and represents the average percentage of time each car spends in a hazardous condition.

Hazard Region 1

The first hazard region considered was based on the measurement of the closing speed between a pair of car-following vehicles and the separation distance between them. The pair would fall within the hazardous region if the separation distance between them was so small that the following vehicle would have to decelerate at a rate greater than 5 ft/sec/sec to avoid hitting the lead vehicle. It was expected that this region would give a measure of those vehicles that made a lane change by cutting in too close to a following vehicle. However, almost no pairs fell within this region so that the hazard ratios computed by using this measure were extremely low. The values computed for this

Figure 10. Number of accidents in ⅕-mile intervals.

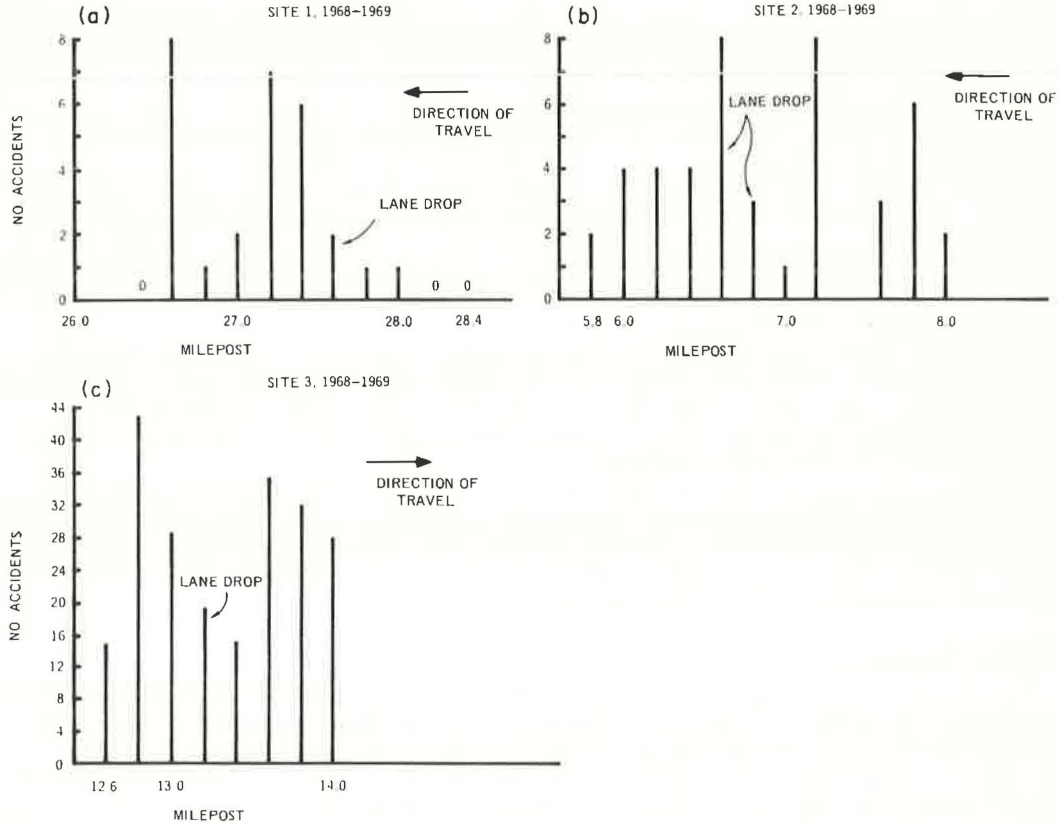
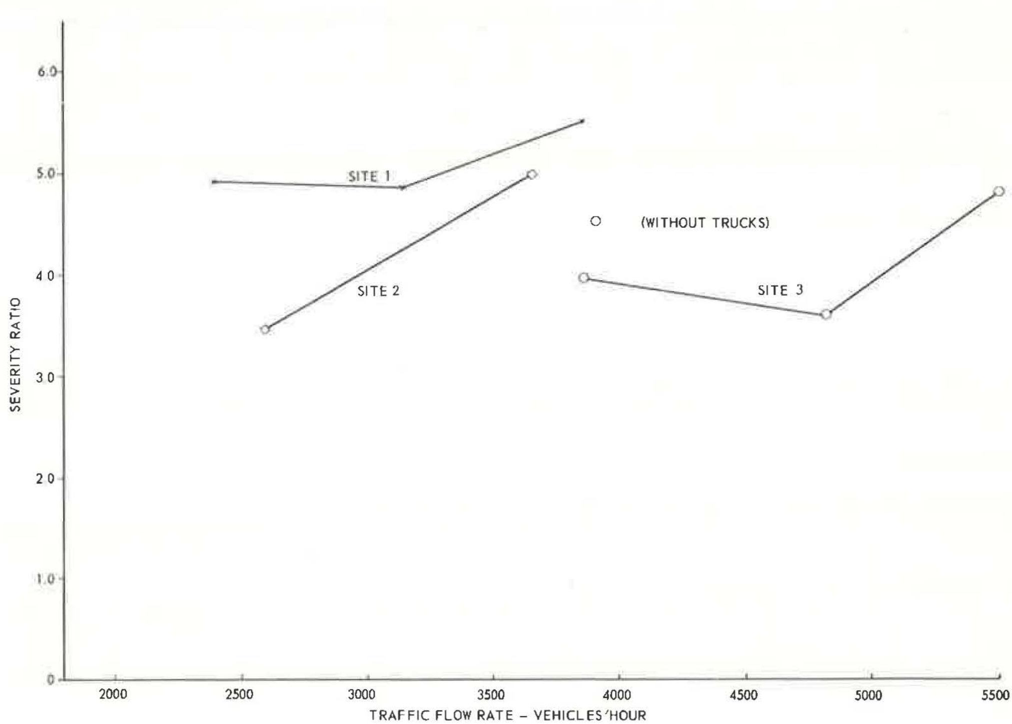


Figure 11. Severity ratio.



measure also fluctuated greatly at different sites and at different traffic flow rates within the same site, so it was decided to discard the measure as a means of evaluating safety effectiveness.

Hazard Region 2

The second hazardous region was defined by considering a driver's response to a rapid braking maneuver of a vehicle in front of his, including an allowance for a lag in his response time. The predicted positions of the 2 vehicles are given by

$$x_1 = x_1 + v_1t + (1/2)\ at^2$$

$$x_2 = x_2 + v_2t + (1/2)\ a(t - T)^2$$

where a is the deceleration rate of both vehicles and T is the response time lag. The hazard region is given by

$$v_2T - (x_1 - x_2 - L) + (1/2a)\ (v_2^2 - v_1^2) > 0$$

where L is the average vehicle length, 17.6 ft. For the current study, a was taken to be 10 ft/sec/sec and T to be 1 sec. Hazard ratio 2 parallels the time criteria discussed by St. John and Glauz (4, 5, 6).

Hazard Region 3

A third region was defined by using the California safe-driving rule, which states that there be at least 1 car-length separation between vehicles for every 10-mph average speed. This can be expressed as

$$v_2/15 = (\Delta x - L)/L$$

which gives the boundary described by the equation

$$v_2 = (15/L)\ (\Delta x - L)$$

The hazardous region corresponding to this safe-driving rule is given by

$$v_2 > (15/L)\ (\Delta x - L)$$

Severity Ratio

The measures discussed above all have what we felt was a common deficiency: They assign equal weights to all potential collisions and do not discriminate between those that have higher probability of occurring or greater potential impact and those that are only marginally hazardous. We, therefore, developed an additional measure known as the "severity ratio," which assigns to each hazardous condition a weighting factor equal to the square of the predicted impact velocities. This weight,

$$w = v_c^2$$

where

w = weighting, and
v_c = projected relative velocity at time of collision,

is a measure of the kinetic energy of the impact. This weighting was implemented for the second hazardous region described above, producing the severity ratio. This ratio is proportionate to the average kinetic energy of the impact of all car-following pairs.

$$s_r = \sum v_c^2 \Big/ \sum_{i=1}^{N} N_i$$

where

$\sum v_c^o$ = all car-following pairs in a hazardous region,

N_i = the number of car-following pairs at time $i\Delta t$, and

N = number of observation instants.

Safety Effectiveness Results

Figure 11 shows the results of the computation of the severity ratio. Hazard ratios 2 and 3 exhibit similar behavior. The approximate standard deviation for the hazard ratios was 0.01 and for the severity ratios was 0.5. When these measures of dispersion were used, the differences between the ratios for the two comparable sites, 1 and 2, were not significant.

INTERPRETATION AND APPLICATIONS

A comparison of the changes in speeds at site 1, where both the pavement and lane striping are dropped on the right, and at site 2, where the pavement is dropped on the right but the stripting merges lane 3 left into lane 4, indicates that traffic in the merging lanes is more unstable at site 2. At site 1, the few vehicles entering the lane-drop area in lane 1 were able to merge smoothly into large gaps in lane 2; but at site 2, because lane 3 and lane 4 traffic was quite dense, the merging movements resulted in reduced headways and subsequently slower speeds. It is likely that restriping site 2 to a configuration similar to that at site 1 would have resulted in an improvement in traffic flow stability through the lane-drop area; however, subsequent addition of a fourth lane and an off-ramp at the downstream end of the lane drop prevented experimental validation of this hypothesis.

Observations of traffic in lanes 3 and 4 at the upstream end of site 2 indicated a problem with lane-drop signing. Although the pavement markings merged lane 3 left into lane 4, the signs informed lane 4 drivers that they should merge right. Although this signing had the effect of distributing merging operations over a longer segment of the freeway than might otherwise have occurred, it also had the effect of requiring more lane changes than were really necessary. The problem of signing for a center lane merged either to the left or right has not yet been satisfactorily resolved, although the California Division of Highways has implemented several ingenious combinations of curves and grades to give the driver the illusion that an outside lane rather than a center lane is being dropped.

Although very few sites at which the pavement was dropped on the left were found in California, and none was found that had high traffic volumes and was also suitable for the aerial photographic techniques used in this study, it would be expected that lane drops resulting from stage construction would frequently be located on the left. Because ramps are usually located on the right and overpasses are usually constructed to span the maximum anticipated width of the roadway, it is easier to add a lane near the median than on the right shoulder. If the traffic flow rates at the left-lane drops are heaviest in the left lanes, it is likely that pavement striping similar to configuration D, merging lane 2 right into lane 1, would provide better traffic service than configuration B, merging the left lane to the right.

The hazard and severity ratios computed for all 3 sites in this study gave no indication of significant differences in the safety effectiveness of the different configurations, and accident analysis of California lane-drop sites corroborated these findings. Although it is premature to state that there are no differences in safety for different lane-drop configurations, it appears that differences in traffic operations are more significant.

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CONTROL OF A FREEWAY SYSTEM BY MEANS OF RAMP METERING AND INFORMATION SIGNS

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A simple ramp-metering strategy is developed for the control of a congested freeway system. If an alternate route can be defined parallel to the freeway, a set of information signs can be erected at choice points. These are positions where a motorist must decide whether to enter the freeway or continue on the alternate route. A strategy for the information sign displays is developed and results in equal travel time on both alternate route and freeway. Finally, it is shown that volume fluctuations can be taken into account if the freeway system is controlled in real time.

•PEAK-PERIOD ramp control is now being used on many freeways as a means of reducing congestion. In a few instances the ramp control method consists of merely closing a ramp for a fixed period each day. In most cases, however, the rate of entry is restricted or metered by means of traffic signals or similar devices erected beside ramps. Within this group, the metering rate in some is determined in real time by the capacity of the freeway immediately downstream from a ramp and by the current freeway volume. This last group is the concern of this paper regardless of precisely when, in a metering cycle, vehicles are released.

If the demand is prevented from exceeding the capacity of any section, which ramp or ramps should be metered: the one immediately upstream or others farther upstream? This problem has been studied by Wattleworth (1, 2) and discussed by May (1).

Wattleworth demonstrated a linear programming approach to maximize total ramp volume, for this will also maximize total freeway travel time for the given input. May showed that this also maximizes total freeway travel. In subsequent comment, May expressed doubt that the same ramp volumes would always maximize both total travel time and total travel criteria.

Both approaches should always lead to the same result. This is shown by solving the same problem as in Wattleworth's paper without using linear programming. This leads to the development of a metering strategy that is then used in conjunction with an algorithm to show the quicker route to the freeway from an alternate route.

METERING STRATEGY

Figure 1 shows the freeway system used in Wattleworth's paper with the numerical order for sections and ramps reversed. Two of the possible critical sections are just upstream from an exit ramp, and the third is just downstream from the last entry ramp. Those are all positions of local maximum demand.

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*Mr. Pretty was with Highway Safety Research Institute, University of Michigan, when this research was performed.

Sponsored by Committee on Freeway Operations.

The following tables were given for A_{jk}, B_k, and D_j. The fraction of vehicles at input j passing section k of the freeway shown in Figure 1 is as follows:

j	k = 1	k = 2	k = 3
1	1.000		
2	1.000		
3	0.949	1.000	
4	0.933	1.000	1.000
5	0.824	0.922	0.969
6	0.519	0.619	0.777

The capacity, B_k vehicle/hour, for each section k of the freeway is

k	B_k
1	6,450
2	6,000
3	5,900

The demand, D_j vehicle/hour, at input j of the freeway is

j	D_j
1	6,800
2	825
3	500
4	450
5	475
6	600

A_{jk} is the decimal fraction of vehicles at input j that pass through section k, B_k is the capacity of the freeway section, and D_j is the hourly demand at input j.

From the tabulations, products can be formed to find the demand for each section D_k^*.

$$D_k^* = \sum_j A_{jk} D_j$$

where only upstream ramps are included in the summation. The values are as follows:

k	D_k^*
1	6,583
2	5,920
3	6,178

Starting with section 1, the section most upstream, there is an excess of demand over capacity of 133 vehicles. Therefore, 133 vehicles must be removed. They should be removed from the next ramp upstream from the section (Avenue C). Otherwise more than 133 vehicles would have to be removed from ramps farther upstream to guarantee that the demand is sufficiently reduced. This is because some vehicles that could be removed from farther upstream were intending to exit at an intermediate exit. By this strategy, the entering ramp volumes are kept at a maximum according to Wattleworth's criterion.

Total travel is also maximized in this method because travel is reduced for only the section of freeway to the next entry ramp upstream. Removal of vehicles from farther upstream would also reduce total travel for those sections without affecting the immediate upstream section. Therefore, May's criterion of maximum total travel is also satisfied by the removal of 133 vehicles from the Avenue C ramp.

The demand at section 2 is already less than capacity; the removal of 133 vehicles from the ramp will also reduce the demand for section 3 to 6,053, leaving an excess of only 153. Therefore, 153 vehicles must be removed and, for the same reasons as before, they should be removed from the next ramp upstream from the section (Avenue F). The ramp volumes X become $X_1 = 6,800$, $X_2 = 825$, $X_3 = 367$, $X_4 = 450$, $X_5 = 475$, and $X_6 = 447$. This solution is identical with Wattleworth's.

Ramp metering would have to be installed at the Avenue C and Avenue F ramps. Because the demand is greater than the metering rate corresponding to the values of X, queues will build up on those ramps. Wattleworth makes no attempt to speculate on what effects there will be on the street system when queues develop.

Figure 1 shows no surface streets. The freeway model actually comes from Wattleworth's earlier paper (2), which names the ramps on the Congress Street Expressway, Chicago. If there is easy surface street access from 1 ramp to the next, then one could expect that many vehicles would travel on the surface streets from near the entrance to the Avenue C ramp to enter the Avenue D ramp without metering. With a long queue on the Avenue C ramp, those motorists could save considerable travel time by diverting.

Because most diverted vehicles would appear at Avenue D, the demand at downstream sections may still be exceeded. This could well mean that ramps with diverted traffic would also have to be metered. In some cases it might even be necessary to increase the amount of upstream metering because of a possible lower limit in the volume entering any ramp.

The pattern of surface streets may be such that it is impossible to estimate the queue length without joining the queue. A set of information signs at the choice points could therefore be erected to tell motorists whether to enter the freeway or to use the surface streets.

OPERATING CONDITIONS FOR INFORMATION SIGNS

From the method used in the previous section, it is possible to identify one or more consecutive entry ramps that require metering to reduce demand. Figure 2 shows a typical freeway system. After n entry ramps, all diverted vehicles may enter at the $(n + 1)$ entry ramp because demand downstream from this ramp is less than capacity. Assume that there is an alternative route with n choice points at which a vehicle may either enter the freeway or continue on the alternate route. After n choice points, the alternate route leads directly to the $(n + 1)$ ramp. It does not matter in a particular case if there are actually 2 or more alternate routes carrying diverted traffic.

From the metering rates, freeway volumes between ramps may be calculated and, hence, average speeds $\bar{v}_s$ corresponding to those volumes. It is assumed that the metering is effective so that $\bar{v}_s \geq v_c$ where v_c is the average speed at capacity. Travel times in the corridor are assumed to be the following:

F_i = freeway travel time, min, from the i th entry ramp to the next entry ramp with metering in operation;
S_i = surface-street travel time, min, from the i th choice point to the next choice point and is independent of the number of vehicles diverted by metering;
R_i = travel time, min, from the i th choice point to the i th entry ramp, excluding a queuing delay on the ramp;
m_i = metering rate, vehicles/min, on the i th ramp necessary to prevent the downstream demand exceeding capacity; and
Q_i = queue length on the i th ramp.

It is assumed that even at capacity a freeway journey is faster than one on surface streets, i.e.,

$$R_i + R_i < S_i + R_{i+1} \tag{1}$$

for i = 1, 2, . . ., n - 1, and

$$F_n + R_n < S_n \tag{2}$$

because the alternate route leads directly to the ramp.

As the queues build up on each metered ramp, the direct route to the freeway becomes less attractive. Eventually,

Figure 1. Wattleworth's freeway.

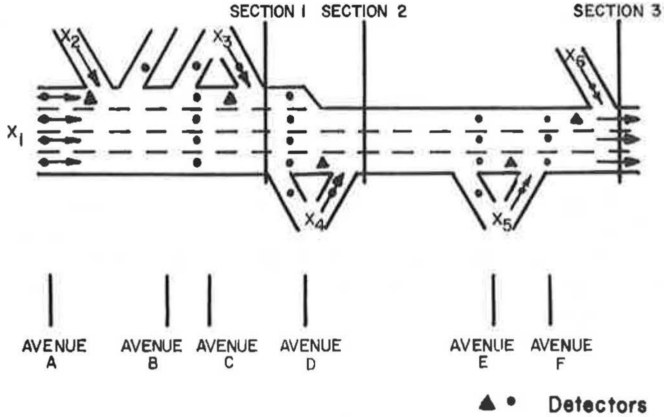


Figure 2. Typical freeway.

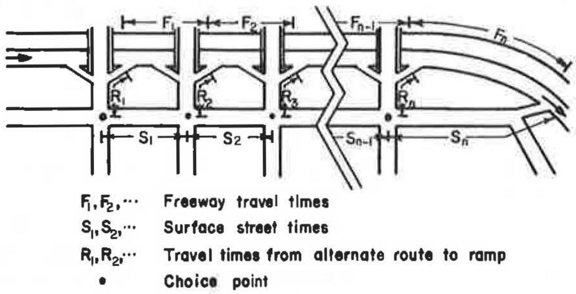


Figure 3. Lodge Freeway.

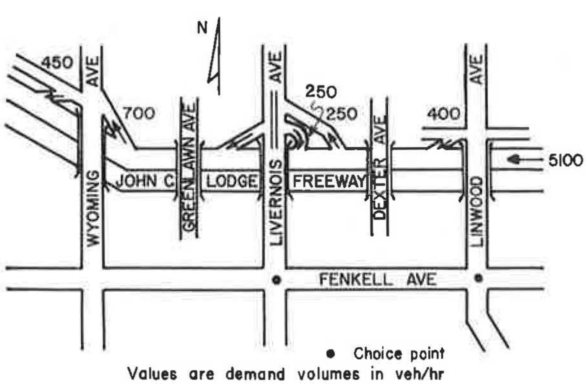
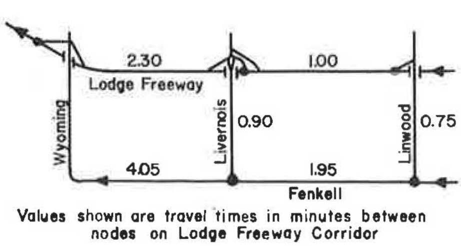


Figure 4. Lodge Freeway travel times.



$$F_n + R_n + Q_n/m_n > S_n \tag{3}$$

and similarly

$$F_i + R_i + Q_i/m_i > S_i + R_{i+1} + Q_{i+1}/m_{i+1} \tag{4}$$

for i = 1, 2, . . ., n - 1.

A set of information signs could indicate the quicker route to the next downstream ramp or choice point from each choice point. If sufficient numbers follow the signs that suggest the use of the alternate route, then surface street and freeway travel times will reach a state where the travel times will be about the same for each route. The condition to attain this state of equilibrium is

$$p_i \geq 1 - 60\ m_i/D_i \tag{5}$$

where p_i is the proportion obeying the sign and D_i is the demand for the ramp, vehicles/hour.

It should be noted that D_i includes any vehicles diverted from upstream ramps. If this condition is not satisfied for any ramp, then an equilibrium queue length cannot be obtained. The lack of an equilibrium queue length at any ramp may be caused by additional diverted vehicles from upstream. More extensive upstream metering will therefore have to be applied so that more vehicles can enter at this ramp.

The equilibrium queue lengths are obtained by solving Eqs. 3 and 4 after the inequalities are removed. They do not have much meaning unless continuous sign state changes occur. It is more realistic to expect that the signs will be adjusted at regular intervals such as every minute. In this case the queue length at the beginning of a minute should determine whether the sign state should be changed. The condition for change is that a hypothetical or test car entering the freeway from the choice point at the half-minute should save time by following the sign when it is pointing to the freeway (green) and lose time when it is pointing to the alternate route (red). An allowance must be made for the expected number of vehicles to arrive at the ramp before the test car. For equal travel times to the nth ramp on the freeway and the alternate route,

$$T_i = \sum_{j=i}^{n} S_j - \sum_{j=i}^{n} F_j \tag{6}$$

so that T_i corresponds to the ramp departure time of the test car. That is measured from the time of departure from the choice point. It is more convenient to omit the ramp subscript i in the remaining notation.

There are 2 sign-switching decisions: the queue length when the sign should change from green to red and vice versa. Those are denoted by Q_{gr} and Q_{rg} respectively and are obtained by equating T from Eq. 6 to the actual departure time of the test car. Thus,

$$T = R + (1/m)\ \left\{Q_{gr} + (D/60)\left[R + 0.5(1 - p)\right] - (R + 0.5)m\right\} + a$$

for a change from green to red and

$$T = R + (1/m)\ \left\{Q_{rg} + (D/60)\left[R(1 - p) + 0.5\right] - (R + 0.5)m\right\} + a$$

for a change from red to green where a is the time to the next departure after the test car's arrival at the ramp; $0 \leq a < 1/m$.

Because T is known from Eq. 6,

$$Q_{gr} = m(T + 0.5 - a) - (D/60)\left[R + 0.5(1 - p)\right] \tag{7}$$

and

$$Q_{rg} = m(T + 0.5 - a) - (D/60)\left[R(1-p) + 0.5\right] \tag{8}$$

Finally, Q_{gr} and Q_{rg} are corrected to the next higher or lower integer respectively. Similar equations can be written for any other decision interval.

If p is the proportion diverting for the demand D, then the signs will point to the freeway (green) for the propertion of time p* given by

$$p^* = \left[60m - D(1-p)\right]/Dp \tag{9}$$

For the remainder of the time, (1 - p*), the sign will point to the surface street.

In this model, it is not necessary for all traffic to have a destination beyond the (n + 1) entry ramp. In fact there could be several exit ramps on the freeway, and many trips will take place only on the surface streets. This is allowed for by reducing the demand at ramp i to include only those vehicles with a destination at least beyond the next freeway exit ramp. Vehicles with destinations closer than that will not even consider entering the freeway.

NUMERICAL EXAMPLE

Wattleworth's Chicago freeway problem is difficult to adapt to this model. One reason is that the next ramp is not necessarily easily accessible from the previous one. For another, if there is diversion downstream the capacity at section 3 will be exceeded. In the present model the freeway ends with a section operating at less than capacity. A suitable example appears to be the following problem from the John C. Lodge Freeway in Detroit (Fig. 3).

The demand volumes (vehicles/hour) are as follows:

Facility	Volume
Freeway	5,100
Linwood entry ramp	400
Livernois exit ramp	250
Livernois East entry ramp	250
Livernois West entry ramp (no metering)	100
Wyoming exit ramp	700
Wyoming entry ramp	450

Critical sections are at Dexter and Greenlawn. At Dexter the demand is 5,500, and the capacity is only 5,400 vehicles/hour. At Greenlawn the demand is 5,600, and the capacity is 5,400 vehicles/hour. It is known, let us assume, that, if a sign is erected at the choice points shown in Figure 3, then as much as 50 percent will divert from the first choice point and 75 percent from the second.

The demand for Dexter can be reduced by removing 100 vehicles/hour from Linwood; the demand at Livernois East will increase to about 345. Another 195 should, therefore, be removed from Livernois East. An additional 30 vehicles have been removed so that a metering rate of an integral number of vehicles can be established. The rates are 5 vehicles/min at Linwood and two vehicles/min at Livernois East.

The equilibrium queue lengths, Q_2 and Q_1, were obtained from Eqs. 3 and 4; equality is assumed. For the travel times shown in Figure 4, $Q_2 = 1.70$ and $Q_1 = 9.75$.

These equilibrium queue lengths are not directly related to the strategy queue lengths for a change in sign state. If Eqs. 7 and 8 are used, these queue lengths are $Q_{gr} = 9$ and $Q_{rg} = 9$ at Linwood and $Q_{gr} = 0$ and $Q_{rg} = 0$ at Livernois. Thus, if the queue length at Linwood at the start of a minute is 9 or more and the sign is green, the sign state should be changed. Similarly, if the queue length is 9 or less, the sign, if red, should be changed to green. At Livernois the sign will immediately turn red and only return to green if the queue length again becomes zero.

The proportion of time the sign is green is, from Eq. 9, 50 percent at Linwood and 13 percent at Livernois. If p* in Eq. 9 had been negative, the development of equilibrium queue lengths would have been impossible and the metering rate at Livernois would have had to be increased. A corresponding decrease in metering rate would be necessary at Linwood. If, in turn, equilibrium could not be obtained at Linwood, the next upstream ramps would have to be metered.

DISCUSSION OF CONTROLS

The alternate route, the freeway, and the streets connecting the 2 form a network with the choice points as origins and the off-ramps as destinations. For any origins and destinations, the ramp-metering and sign strategies lead to a compliance with Wardrop's principles (3):

1. The journey times on all routes actually used are equal and less than those that would be experienced by a single vehicle on any unused route, and
2. The average journey time is a minimum.

Actually the satisfaction of the second principle is conditional on the first. If, with equal travel times, more drivers for some reason chose to travel on the surface street, a few could travel on the freeway and enjoy a much faster travel time. Average journey time would be less than the equilibrium case. However, that would violate the first principle.

This problem arises because a constant travel time has been assumed on the alternative route, which has been given an inifinite capacity. Neither of these assumptions is strictly true, although, for urban streets with only a small additional volume, they are good approximations. In a recent paper, Potts (4) also investigated Wardrop's principles for this case.

The metering rate and information sign strategy have been derived on the assumption that all volumes are constant. There are, of course, short- and long-term fluctuations. Fortunately fluctuations in entry-ramp volumes can be partly controlled by the information signs. For example, if there is a fall in the demand for a ramp, the information signs in trying to achieve equilibrium queue lengths automatically will try to direct more traffic to the ramp by staying green longer. However, a change in freeway volume may require a change in metering rates. Similarly, there could be an accident on the freeway, and that would require drastic and immediate reductions in upstream metering rates and a shift to the red state for upstream information signs. Finally, there could be a change in exit-ramp usage at least during a peak period.

To take into account fluctuations over time requires that the control of the signals and signs take place in real time. A set of freeway and ramp detectors, including queue detectors, would be required, and connection would have to be established with a digital computer to determine the metering rates. Communication to the field would then be necessary to change the sign states and metering rates or directly the ramp signal states. If the interval for sign state adjustment is to be 1 min, the metering rates should be altered less frequently so that the alternative travel times can be accurately anticipated. No detectors can give current origin and destination patterns (entry ramp by exit ramp). Hence, those data would have to be obtained by an independent survey. For use in real-time ramp control, it would be sufficient to update this information at widely spaced time intervals such as ½ or 1 hour.

CONCLUSION

A rational method has been presented for determining the metering rates and information sign strategies for the control of a freeway section. The data required are an origin-destination table for entry ramps and exit ramps, freeway and entry-ramp demand, and freeway capacity. The method could be applied in real-time freeway control projects or used in a theoretical or simulation study to measure the effectiveness of information signs in reducing travel time.

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DISCUSSION

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This paper deals with 2 different aspects of the freeway control problem. The first involves the problem of management control and the apportionment of right-of-way on those sections of the freeway for which demand exceeds capacity. The second involves the development of a dynamic control algorithm with which to implement motorists' diversion on a real-time basis.

Freeway Management

The management strategy presented in this paper appears to be optimal in the same sense as the control algorithm developed by Wattleworth. In view of this and the relatively simple logic on which it is structured, this control algorithm is extremely attractive. It offers a combination of logical simplicity and computational ease that makes it worthy of experimental evaluation.

Development of the management control algorithm is based on deterministic arguments regarding average values. To reflect concern regarding the variability of demand, the author recognizes that the production resulting from a fixed-rate, ramp-metering operation must of necessity be less than the fixed-metering rate. The paper suggests that the metering rate be set slightly higher than the desired ramp volume. Presumably, readjustment of this form would result in a production rate more closely aligned with that determined by the management policy.

Analysis of the ramp-metering control problem has revealed more suitable means with which to achieve control of the ramp-metering production rate. In particular, responsive metering that reflects the upstream approach volume on the freeway provides a better solution to this problem. In addition, the resulting correlation between entering traffic and the main stream yields a less turbulent output stream in which significant reductions in the accident rates have been observed.

It is quite apparent that the control algorithm presented in this paper is suitable for the solution of the management control problem. It provides for a simple, logical determination of the proper right-of-way apportionment. However, use of a responsive ramp-metering system designed to produce the desired average entrance rates appears desirable. Only demand-responsive systems can provide for proper consideration of freeway and ramp demand variations.

Informational Signing

Utilization of a control algorithm for the periodic resetting of a diversionary signing system located upstream from an entrance ramp is basically sound. The concept of periodic readjustment is consistent with the control concepts utilized in the apportionment of right-of-way at intersections. It constitutes a reasonable restriction. Undoubtedly, details of geometry, sign location, and volumes to be diverted heavily color the potential effectiveness of the system.

The control algorithm with which the author proposes to determine whether the state of the diversionary signs should be switched raises several questions. For example, the control algorithm assumes a relatively stable approach volume at the diversionary control sign point. This is undoubtedly questionable if the sign is located within a network of signalized intersections in which a fixed cycle length and platoons of vehicles exist. In addition, the decision mechanism requires measurement of the queue length. Even if this quantity remains constant (in spite of its dependence on travel time differences), various questions of measurement accuracy and system dynamics arise. For example, the percentage of the time that the diversionary sign directs traffic to the freeway may well become negative as a result of small changes in motorist compliance. Although this in itself does not invalidate the algorithm, it does identify an important point regarding the basic limitations of diversionary control systems.

Summary

Although there is a variety of questions that one can pose regarding the logical implementations of the control algorithms presented in the paper, one should not lose sight of the fact that the basic premise in both instances is sound. The author has raised significant points regarding 2 aspects of the solution of the freeway control problem. In so doing, it is to be expected that additional questions that require investigation will be raised.

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The congestion that prevails on many freeways is caused primarily by the demand exceeding the capacity of the freeway system. In many instances, unused capacity exists not only at selected points on the freeway downstream of a bottleneck but also on alternate routes in the freeway corridor. Concerning the matter of unused capacity, the hands of the traffic engineer in charge of freeway operation are tied unless he can communicate with the vehicle drivers. More important, any communication with the driver must be accurate, timely, and clearly understood. The author of this paper deals with the problems of providing accurate information to the prospective freeway user. This discussion further considers the problems of providing timely information.

One of the potential uses of driver communications cited by the author is to minimize the total travel time in the freeway corridor. This is primarily accomplished in the following ways:

1. By directing some traffic off the freeway to alternate routes that have available capacity,
2. By directing some approaching traffic that would normally use the freeway to alternate routes, and
3. By improving communications so that less trial and error is involved in maneuvers 1 and 2 and that travel time is further reduced.

To proceed a step further, it seems appropriate to outline the basic elements of any communication system. Those elements include detection, transmission, data processing, and display. The problem considered by the author in this paper is centered around the detection and the data processing elements.

One inviting aspect of the paper is the explicit underlying assertion that communication systems must be a part of a workable freeway control scheme, preferably one that operates in real time with respect to current traffic conditions. In presenting a control scheme, the author develops a simple strategy based on previous work done by Wattleworth. It will suffice to say that the author's strategy duplicates the numerical solution presented by Wattleworth.

The author further develops several equations directed toward establishing operating conditions for information signs. The basic objective is to provide a state of equilibrium at which time the surface street and the freeway travel times will be about the same for each route. The conditions for this state are based on, among others, the following assumptions: the ability to estimate queue length on each controlled ramp and the ability to assign a metering rate that will be realized during short periods of time.

Because of the microscopic nature of the author's information sign-change model, the first assumption (i.e., queue-length estimation) creates several practical problems that from the discussant's experience are beyond a cost-effective solution within the current state of the art in vehicle detection. Indeed, many freeway control models now in use are operating at less than optimum because of variations in performance found in nearly all vehicle-detection systems. This is pointed out to suggest a close look at the author's model with regard to model sensitivity to queue-estimation errors.

The second assumption considered in this discussion is an interesting one from the standpoint of applying the control scheme presented by the author to a real-time control situation. It has been the discussant's experience that during relatively long periods of time, that is to say 15 to 30 min, metering rates can be assigned with high expectation that the rate will be met. However, in a control scheme where metering rates may change as often as each minute, unless those rates are closely related to the dynamic merging conditions at each ramp, one cannot predetermine absolutely the number of vehicles that will actually enter the freeway. It may be more appropriate to base the control on the probability of achieving the desired metering rate. The point being made is that the actual metering rate will be less than that assigned because of poor merging conditions, responsiveness of drivers to the ramp signal, and vehicles stopping in the merging area. Again, this consideration is made important because of the microscopic nature of the author's model.

In summary, this discussion of the paper supports the author's assertion for the need of an integrated freeway control system that includes communication with the vehicle drivers, presents some limitations of current detection systems with respect to the author's model, and gives some of the discusser's experience in achieving assigned metering rates on controlled freeway ramps. The delineation of the problems given above is meant not to detract from the model developed, but to point out some of the factors that will have to be considered in evolving a practical information system for corridor traffic.

AUTHOR'S CLOSURE

Both discussants have raised the important questions of the achievement of assigned metering rates and the measurement of queue lengths. It is convenient to consider first the frequency of metering-rate review. That should be less often than review of sign-state changes because the algorithm for sign-state changes assumes a stable metering rate and, depending on the relative position of the several ramps, the calculation of the demand for the bottleneck and, hence, the metering rates require a stable origin and destination pattern that can hardly be meaningful during a period of less than 15 min. Incidentally, in response to a point made by Yagoda, the frequency of sign-state changes should be closely related to the local signal cycle times.

The preset metering rate must be subject to a lower bound to allow for signal disobedience and an upper bound because of merging difficulties. Within those limits, it should be possible to release exactly the number of vehicles required. Because the methods of release from metering signals have been fully investigated elsewhere, the scope of this paper has been limited to the determination of the necessary preset rate.

Given a stable metering rate and demand pattern, the sign-state changes depend on the queue length. It is acknowledged that available detection systems cannot reliably measure this. Fortunately, it is much easier to detect whether a queue does or does not reach a particular detector. Occupancy rises sharply from a low value to a high one. It would thus be feasible to place a pair of detectors at the critical queue lengths for sign-state changes from green to red and vice versa. An average value of jam density would be assumed. Other pairs of detectors could be placed for different metering rates and demands. In a particular case, however, 1 or 2 pairs of detectors would probably be sufficient for normal peak-period conditions.

In answer to Yagoda's final point, the algorithm assumes a stable motorist response to the signs. If, during a period of months, the percentage of time that the sign directed motorists to the freeway became zero and the queue length did not reach equilibrium, then the metering rate would have to be increased. As a result, the upstream metering rates could be affected.

ESTIMATION OF SPEED FROM PRESENCE DETECTORS

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Presence detectors are currently in use in surveillance and ramp control projects in several cities in the United States. The data obtained from those detectors are usually processed to produce traffic volumes and occupancies. By making certain assumptions, it is possible to obtain estimates of the average (space mean) speed of a group of vehicles passing over a single sensor. The simplest estimators (of the form length divided by detector on time) are biased. The purpose of this paper is to develop unbiased, minimum mean square error estimates of speed. These new speed estimators differ from the simplest estimators in that on-times associated with the passage of individual vehicles are processed separately. Although individual speeds cannot be determined very accurately, the estimators developed here are capable of determining the average (space mean) speed of, say, 20 vehicles to within 4 mph. A freeway traffic simulation is employed to illustrate the superiority of the estimators derived here.

• THE SURVEILLANCE of traffic on freeways in several cities (Los Angeles, Chicago, Houston, and others) is based on measurements acquired from presence detectors. These devices consist of a sensor (either a magnetic loop or a sonic instrument) and a detector that converts the changes of a signal produced when vehicles pass through the influence zone of the sensor into an on-off type of signal. The presence detector measurements are typically converted into estimates of volume (vehicles/hour), occupancy (fraction of time the sensor is activated, usually expressed in percent), and density (vehicles/lane/hour). It is the purpose of this paper to examine the use of presence-detector measurements for the unbiased estimation of individual and space mean vehicle speeds.

If the on-time measured by the presence detector for the passage of an individual vehicle is t, the speed of that vehicle can be determined from

$$V = L/t \tag{1}$$

if L, the effective length of the vehicle and the sensor combined, is available. This effective length, however, varies from vehicle to vehicle (and for a fixed vehicle, possibly from sensor to sensor) so that at best one only has available a distribution of effective lengths. This distribution is to be identified in experiments under way for the specific hardware used on Los Angeles freeways. A description of the experiments is contained in the Appendix. For our purposes, we will assume that the mean length m_L and standard derivation σ_L are known.

In addition to the uncertainty in the effective length, the presence signal is usually sampled, for example, every T seconds, so that the on-time is quantized in the form $t = nT$ where n is an integer. This, of course, leads to additional errors because it is possible for this quantized measurement of time to be in error by nearly a full sampling

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interval. If a very high sampling rate is used, a variation of one in the number of presence pulses is insignificant. However, the current trend is toward lower sampling rates because of the cost of transmitting and processing the large amounts of data generated by high sampling rates. Significant errors in the measurement of time will exist with sampling rates on the order of 15 to 20 samples per second. In the next section, the statistics of the number of presence pulses are determined.

Because an individual vehicle speed cannot be precisely measured by a presence detector, one can at most ask for an unbiased estimate. An unbiased estimate, in the instance of estimating vehicle speed, is one that has the property that a collection of vehicles passing over the sensor would generate a set of estimates whose average would tend to the average of the true speeds as the size of the collection is increased. The speed estimator $\hat{v} = m_L/nT$ is not unbiased; the unbiased estimator is determined in a later section.

In traffic surveillance, the average speed of the vehicles on the road is usually of more interest than a collection of estimates of individual speeds. This space mean speed, under conditions of space and time homogeneity of traffic conditions (1), can be determined from the speeds of vehicles as they pass a fixed point on a road from the relation

$$v_s = N \Bigg/ \left[\sum_{i=1}^{N} (1/v_i)\right] \tag{2}$$

where v_i, $i = 1, \ldots, N$ are the sequence of individual speeds.

If one uses the estimate

$$\hat{v}_i = m_L/n_i T \tag{3}$$

for the ith vehicle (which generates n_i presence pulses), one is naturally led to estimate space mean speed by

$$\hat{v}_s = Nm_L \Bigg/ \left(T \sum_{i=1}^{N} n_i\right)$$

If these measurements are taken during the time period MT, occupancy is defined by

$$\theta = \left(\sum_{i=1}^{N} n_i\right) \Bigg/ M \tag{4}$$

With this notation, the space mean speed estimate becomes

$$\hat{v}_s = (N/MT)\,(m_L/\theta) \tag{5}$$

Note that N/MT is the volume during this period of time. This estimate of space mean speed is also biased; the unbiased estimator is determined in a later section.

In the last section, the various estimators are compared by using data obtained from a car-following simulation of freeway traffic.

The Appendix contains a description of 2 experiments, which had 2 primary results. First, values for m_L and σ_L for the magnetic loop sensors and associated detectors in use on Los Angeles freeways were determined. Second, it was determined that, as long as vehicles stay within 12-ft lanes, centered magnetic loops, 6 by 6 ft, produce presence signals that vary by less than 2 percent for any fixed speed.

DISTRIBUTION OF NUMBER OF PRESENCE PULSES

The number of presence pulses that will be generated by a vehicle passing over the sensor is given by

$$n = [(Lf/v) + e] \tag{6}$$

where L is the combined vehicle and sensor effective length, f is the frequency of sampling, v is the vehicle speed, e is a random variable uniformly distributed on (0,1/f) that represents the random arrival time, and [x] is the greatest integer less than or equal to x. With m defined by

$$m = Lf/v \tag{7}$$

it is readily seen that the passage of this vehicle will generate either m or m + 1 presence pulses. Noting the uniform distribution for e, one finds that

$$p(n = m \mid L,\ v) = m + 1 - (Lf/v) \tag{8}$$

and

$$p(n = m + 1 \mid L,\ v) = (Lf/v) - m \tag{9}$$

We shall employ the notation $p(n|V)$ to denote this probability, with the dependence on L suppressed in the notation. Defining the set

$$A_n = \{v\colon\ [fL/(n + 1)] < v \le (fL/n)\} \tag{10}$$

one also has

$$p(n|v) = \begin{cases} n + 1 - (Lf/v) & \text{for } v \in A_n \\ (Lf/v) - n + 1 & \text{for } v \in A_{n-1} \\ 0 & \text{for } v \notin A_n \cup A_{n-1} \end{cases} \tag{11}$$

ESTIMATION OF INDIVIDUAL VEHICLE SPEEDS

Suppose that the measurements $y_1,\ y_2, \ldots,\ y_N$ are available to estimate the random variable x, e.g., by means of

$$\hat{x} = g(y_1,\ y_2, \ldots,\ y_N) \tag{12}$$

The mean square error for this estimator is given by

$$E\ \{[g(y_1,\ y_2, \ldots,\ y_N) - x]^2\} \tag{13}$$

(E { } denotes the expected value.) It is easily shown (2) that the minimum mean square error estimator is given by the conditional expectation

$$\hat{x} = E(x \mid y_1,\ y_2, \ldots,\ y_N) \tag{14}$$

Moreover, this estimator is unbiased.

In this section, we determine the unbiased speed estimator

$$\hat{v}(n) = E(v|n) \tag{15}$$

This estimator is readily obtained if the conditional probability density function $p(v|n)$ is known. From Bayes' formula

$$p(v|n) = p(n,\ v)/p(n) \tag{16}$$

and the 2 relations

$$p(n,\ v) = p(n|v)p(v) \tag{17}$$

$$p(n) = \int p(n,\ v)dv \tag{18}$$

it can be seen that $p(v|n)$ can be found if $p(v)$ is specified, for $p(n|v)$ was determined in the previous section.

Noting that $p(n|v)$ for $v \notin A_n \cup A_{n-1}$, we see that only a portion of the probability density $p(v)$ is required. In fact, over the limited domain $A_n \cup A_{n-1}$, we shall take $p(v)$ to be constant, i.e.,

$$p(v) = k \quad \text{for } v \in A_n \cup A_{n-1} \tag{19}$$

Then from Eqs. 11, 17, 18, and 19, one finds

$$p(n) = kfL \log [(n^2/(n^2 - 1)] \tag{20}$$

And then, using Eq. 16 we have

$$p(v|n) = \begin{cases} \{1/fL \log [n^2/(n^2 - 1)]\} \; [n + 1 - (Lf/v)] & \text{for } v \in A_n \\ \{1/fL \log [n^2/(n^2 - 1)]\} \; [(Lf/v) - n + 1] & \text{for } v \in A_{n-1} \\ 0 & \text{for } v \notin A_n \cup A_{n-1} \end{cases} \tag{21}$$

(Note that k does not appear in this result.)

From Eq. 21, it is an easy matter to determine the desired estimator by integration.

$$\begin{aligned} \hat{v}(n) = E(v|n) &= \int_V vp(v|n)dv \\ &= fL/n \; \{1/(n^2 - 1) \log [n^2/(n^2 - 1)]\} \end{aligned} \tag{22}$$

Note that fL/n represents the standard speed estimator (Eq. 3); to identify the effect of the term in brackets, it can be expanded in a series to yield

$$\hat{v}(n) = fL/n \; [1 + (1/n^2) + 0 \; (1/n^4)] \tag{23}$$

where $0(x)$ denotes a term with the property that $0(x)/x$ tends to a finite limit as x tends to zero.

Throughout these calculations L was considered to be fixed. Averaging over the population of lengths, L is merely replaced by the mean length m_L so that there is no bias introduced by the variability of lengths about the mean.

The minimum mean square estimate of the time mean speed of a sequence of N vehicles that produces the sequence $n_1, n_2, \ldots, n_N$ of presence pulses can be expressed as

$$\hat{v}_t = 1/N \sum_{i=1}^{N} E(v_i | n_1, n_2, \ldots, n_N) \tag{24}$$

Because most of the information concerning the speed of the i th vehicle is contained in n_i, we make the approximation

$$E(v_i | n_1, n_2, \ldots, n_N) \simeq E(v_i | n_i) \tag{25}$$

so that the time mean speed estimator becomes

$$\hat{v}_t = 1/N \sum_{i=1}^{N} E(v_i | n_i) \tag{26}$$

ESTIMATION OF SPACE MEAN SPEED

Space mean speed, under conditions of time and space homogeneity of traffic conditions (1), can be determined from the sequence of speeds measured at a fixed point in space during a period of time by the formula

$$v_s = N\Big/\left(\sum_{i=1}^{N} 1/v_i\right) \tag{27}$$

where v_i, $i = 1, \ldots, N$ are the speeds of the N vehicles. In fact, it is an easier matter to determine an estimate of $1/v_s$. To correct the 2 estimates, we define

$$u_s = 1/v_s = 1/N \sum_{i=1}^{N} u_i \tag{28}$$

where

$$u_i = 1/v_i \tag{29}$$

Suppose $\hat{u}_s$ is the estimate of u_s based on presence pulse data and $\sigma^2_{u_s}$ is the variance of this estimate; i.e.,

$$\hat{u}_s = E(u|n_1, n_2, \ldots, n_N) \tag{30}$$

$$\sigma^2_{u_s} = E[(u_s - \hat{u}_s)^2|n_1, n_2, \ldots, n_N] \tag{31}$$

If $\sigma_{u_s}/\hat{u}_s << 1$, it is readily shown that

$$E(v_s|n_1, n_2, \ldots, n_N) = 1/\hat{u}_s \{1 + [\sigma^2_{u_s}/(\hat{u}_s)^2] + 0(1/\hat{u}^4)\} \tag{32}$$

We shall use the first 2 terms in the parentheses for our space mean speed estimator.

It remains to determine $\hat{u}_s$ and $\sigma^2_{u_s}$. As for the time mean speed estimator, we shall make the approximation

$$E(u_i|n_1, n_2, \ldots, n_N) \simeq E(u_i|n_i) \tag{33}$$

so that we shall take

$$\hat{u}_s(n_1, n_2, \ldots, n_N) = 1/N \sum_{i=1}^{N} E(u_i|n_i) \tag{34}$$

Similarly,

$$\sigma^2_{u_s} \cong 1/N^2 \sum_{i=1}^{N} \sigma^2_{u_i} \tag{35}$$

where

$$\sigma^2_{u_i} \cong E\{[u_i - E(u_i|n_i)]^2 \; n_i\} \tag{36}$$

Our computations can be completed once $E(u_i|n_i)$ and $\sigma^2_{u_i}$ are determined.

Now (suppressing the subscript notation),

$$E(u|n) = \int (1/v)\, p(v|n) dv$$

which is readily determined to be

$$E(u|n) = (n/fL) \left(\{\log[(n + 1)/(n - 1)]\}/\{n \log[n^2/(n^2 - 1)]\}\right) - 1 \tag{37}$$

$$= (n/fL)[1 - 1/3n^2 + 0(1/n^4)] \tag{38}$$

This has been obtained under the same assumptions as employed in the previous section [e.g., p(v) is constant].

Similarly, one finds

$$E(u^2|n) = (1/fL)^2 \{1/\log[n^2/(n^2 - 1)]\} \tag{39}$$

$$= (n/fL)^2 [1 + (1/2n^2) + 0(1/n^4)] \tag{40}$$

so that

$$\sigma_u^2 = (n/fL)^2 [(7/6n^2) + 0(1/n^4)] \tag{41}$$

The space mean speed estimator is then defined by Eqs. 32, 34, 35, 38, and 41. The assumption that $\sigma_{u_s}/\hat{u}_s << 1$ is justified for even moderate values of n_i.

Up to this point L has been fixed. To account for the variability in L requires that an expectation with respect to L be taken. In contrast to the situation with the time mean speed, L cannot be simply replaced by m_L; in fact, we require E(1/L) as L appears in the denominator of $E(u_i|n_i)$. Using the same reasoning that led to Eq. 32, one has

$$E(1/L) = 1/m_L [1 + \sigma_L^2/m_L^2 + 0(1/m_L^4)] \tag{42}$$

if one assumes $\sigma_L/m_L << 1$. The first 2 terms of those within the brackets will be employed. This correction need not be made in calculating $\sigma_{u_i}^2$ because the net correction is of the same size as terms that are already being ignored; i.e., L can be replaced by m_L.

In the preceding discussion, the speed distribution was taken to be uniform in the limited range of interest. There are instances, to be discussed in the next section, in which a ramp function more closely approximates the situation. If the distribution of speeds over the range of interest is assumed to be

$$p(v) = \begin{cases} k(v_b - v) & v \leq v_b \\ 0 & v > v_b \end{cases} \tag{43}$$

one finds in a similar manner

$$E(u|n) = (n/fL) \Big(\{(1 + m/n) \log [(n^2 - 1)/n^2] + 1/n \log [(n + 1/n - 1)]\}/ \{\log [(n^2/n^2 - 1)] - m/[n(n^2 - 1)]\}\Big) \tag{44}$$

and

$$E(u^2|n) = (1/fL)^2 \Big(\{1/m - n \log [(n^2 - 1)/n^2] - \log [(n + 1)/(n - 1)]\}/ \{1/m \log [n^2/(n^2 - 1)] - 1/[n(n^2 - 1)]\}\Big) \tag{45}$$

where we have defined

$$v_b = fL/m \tag{46}$$

DISCUSSION OF ESTIMATES

The estimation of individual vehicle speeds or the space mean speed should be performed on individual vehicle data rather than over some arbitrary fixed sampling period because of the possibility in the latter instance of terminating a sample while a vehicle is directly over the sensor. For example, if the passage of the vehicle is sensed by its arrival at the sensor, the vehicle would be treated as an exceptionally fast vehicle if the sampling period terminated while that vehicle was still over the sensor. It is desirable to avoid the introduction of additional error due to the sampling procedure.

The sampling period determines the number of vehicles whose passages contribute to the time mean or space mean speed. Because the new estimators derived here are all of the form of conditional expectations (to within the approximations made in the 2 previous sections), the expected values of the estimates are the true means, regardless of the sample size. However, the sampling period does affect the variance of the estimates. Precisely for time mean speed, and approximately for space mean speed, the variance of the estimates for N vehicles is 1/N times the variance of the estimate of the speed (or reciprocal speed) for an individual vehicle. Hence, a longer sampling period leads to estimates with smaller variance. If the normal flow is 1,200 vehicles/hour, the variance for the mean speed of a 1-min sample would be about 1/20 of the variance of the estimate of an individual vehicle speed.

In addition to estimator variance, the choice of the sampling period depends on the homogeneity of the traffic flow and the required timeliness of the data. If the estimates are to be used for control purposes, they must be timely. Accurate estimates depicting roadway conditions long past are of little value for control. For these purposes, a 1-min sampling period was employed in the Eisenhower Expressway Study (3) and has been selected for the current surveillance project in Los Angeles (4).

It is evident that the sampling rate affects the accuracy of speed estimators. For example, the variance of the unbiased speed estimator (Eq. 22) is given by

$$\sigma_v^2 = (fL/n)^2 \left((3n^2 - 4)/\left\{3(n^2 - 1)^2 \log \left[n^2/(n^2 - 1)\right]\right\}\right) \tag{47}$$

$$= (fL/n)^2 \left[(1/6n^2) + 0\,(1/n^4)\right] \tag{48}$$

Figure 1 shows the standard deviation σ_v with L taken to be 28 ft for sampling rates of 15, 30, and 45 samples/sec. The standard deviation of the error due to the sampling procedure is halved by doubling the sampling rate.

The precise form of the unbiased estimator depends on the actual velocity distribution on the roadway. Real roadways nearly always have a speed limit on them. This causes a truncation of velocity distribution in the region of this speed limit because there are fewer vehicles traveling above this limit than below it. It is possible to pick a speed at which no vehicles are traveling. This condition can be represented by the negative ramp distribution case.

At intermediate speeds the vehicle speeds are more uniformly distributed with some cars going slower than the average and others faster. This region can be represented by a uniform distribution. At low speeds there are more vehicles going faster so that this region can be represented by a positive ramp distribution. Figure 1 shows that the standard deviation of the error gets very small at low speeds so that the low-speed effect will be neglected. The bias caused by the uniform and negative ramp distribution is shown in Figure 2. The unbiased estimator depends on the velocity distribution, and the differences become significant at the higher speeds. An unbiased estimator must consider the effects of the different velocity distributions at the intermediate and high speeds. The selection of the wrong distribution at the low velocity end does not introduce major errors into the system. However, the opposite is true at the upper velocity boundary.

The distribution of vehicle lengths is a major source of variability in the estimate of individual vehicle reciprocal speeds. The effects of the distribution of lengths on the error of the estimate of space mean speed can be reduced by increasing the vehicle's effective length or by decreasing the variance of the car lengths. Because actual vehicle lengths are fixed by the automobile manufacturers, this means that longer sensors reduce the bias due to vehicle length distributions. Similarly, if vehicles can be separated into classes by length, the bias due to the length distributions can be reduced by reducing the standard deviation of lengths possible for each class.

SIMULATION AND EVALUATION OF ESTIMATORS

Four Estimators

There are 2 basic sources of error in the estimation of space mean speed by means of the estimator specified by Eq. 5. The first is due to the method of counting vehicles;

the second is due to the distribution of vehicle speeds and lengths. Four estimators will be described and evaluated here by means of a car-following simulation of traffic. The first, labeled estimator 1, is defined by Eq. 5. The count of vehicles N is based on the number of vehicles arriving at the sensor during the sampling period.

Estimator 2 differs from estimator 1 in that consideration is given to the possibility that a vehicle may be over the sensor at the time the sampling period begins or ends. This estimator uses the average of the counts for arrivals and departures to obtain the vehicle count. If the sum of the arrival and departure counts is even, then the estimates derived from estimators 1 and 2 are identical.

The estimates of space mean speed obtained by estimator 3 exclude data obtained from vehicles that have not completely passed over the sensor in the sampling period. This vehicle is included in the next sampling period. Thus, the sampling period is not necessarily the same for consecutive periods.

Estimator 4 includes corrections for the bias due to the speed and length distributions as described by Eqs. 32, 34, 35, 38, and 41. Otherwise, it is of the same form as estimator 3 so that no errors are incurred by inaccurate vehicle counting.

The sequence of estimators described is successively more sophisticated so that it would be expected that the estimates obtained from estimator 2 are better than those obtained from estimator 1 and so on.

Estimator Evaluation

The estimators described above were evaluated by comparing the space mean speed of a set of vehicles passing over the sensor (as defined by Eq. 2) with the 4 estimates of the same quantity determined from the sensor data. The simulation experiment was set up so that all of the estimators operated on the same sensor data. Thus, there is no question of sample differences causing variations in the accuracy of the estimates. All of the differences in the estimates are due strictly to the estimating procedures.

The sensor data were generated by a freeway and sensor simulation of the car-following type (5) in preference to acquiring real data from an actual roadway because it is impossible to control and accurately measure all of the pertinent variables in a real system. Monte Carlo freeway simulation provides a convenient means to examine the accuracy of various estimating schemes because the precise characteristics and performance of vehicles within the simulation can be readily determined and compared against the estimates derived from the sensor outputs. The effects of variables such as loop field variation, detector sensitivity, vehicle lane position, and vehicle height have been removed so that they do not mask the variations in the estimates associated with the vehicle counting and the vehicle and length distributions. In addition, the traffic was restricted to 1 lane to avoid problems due to lane changing over the sensors.

Simulation Results

The performance of the estimators was measured in terms of the mean difference between the estimated and actual space mean speed and the variance of these differences.

Sensor Experiments

The differences between the actual and estimated space mean speeds are given in Tables 1 and 2. The first tabulation gives the mean and variance of the error for each estimator and represents an external view of the experiment. The second tabulation gives the errors by vehicle. Each vehicle is sensed by a count of n_1. The errors of all vehicles that caused a count of $n_1 = m$ were tabulated together so that a posterior velocity distribution and the mean and variance of the errors associated with each element of the distribution could be determined. This provides an internal view of the experiment. Table 1 gives a comparison of the alternate estimators; Table 2 gives the data to determine causes of variation between the predicted and observed results.

The average error and the standard deviation of the error are given in Table 1. The question of bias is examined first. The null hypothesis of zero bias was tested at the 5, 1, and 0.1 percent levels if significant. A 2-sided test was used because the hypothesis of zero bias was examined. The hypothesis of zero bias is accepted for estimator

Figure 1. Effect of sampling rate.

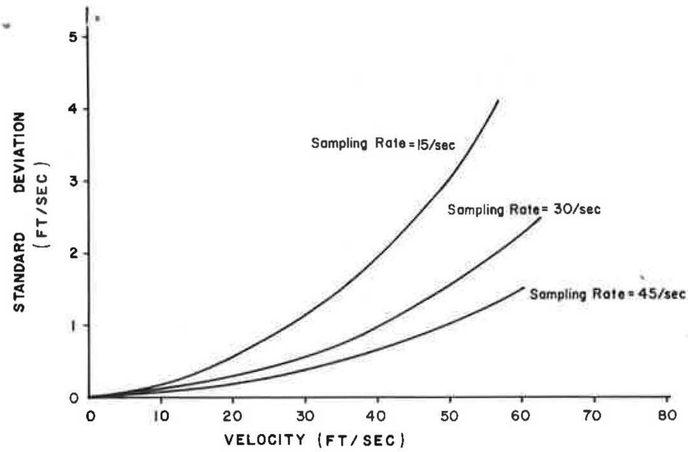


Figure 2. Estimators of 1/v.

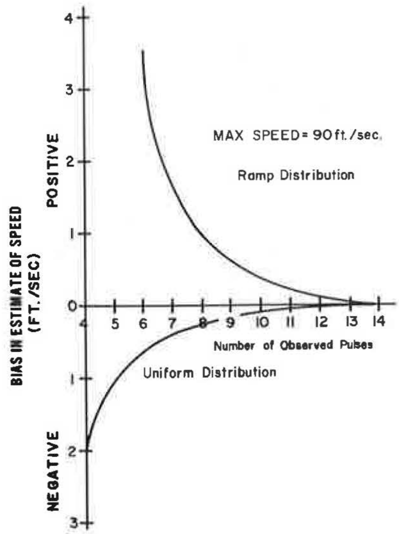


Table 1. Space mean speed estimation errors.

Estimator	Mean	Standard Deviation	Estimator	Mean	Standard Deviation
1	0.592	0.146	3	0.491	0.095
2	0.522	0.118	4[a]	0.179	0.078

[a]The effect of the (σ_μ^2/n_μ^2) term was not included.

Table 2. Distribution of estimation errors for individual reciprocal speeds.

Number of Presence Pulses	Number of Samples	Actual Reciprocal Average Speed (ft/sec)	Estimator 1	Estimator 4
4	2	84.317	105.0	101.0
5	6	77.453	84.0	81.5
6	19	68.410	70.0	70.0
7	13	58.159	60.0	59.8
8	6	52.586	52.5	52.3
9	5	47.670	46.7	46.4
22	1	16.330	19.1	18.9
23	2	17.401	18.26	18.1
24	1	18.200	17.5	17.3
25	5	16.238	16.8	16.6
26	3	16.654	16.2	16.0
27	2	16.472	15.6	15.5
28	1	16.780	15.0	14.9
29	1	15.580	14.5	14.4

Figure 3. Schematic diagram of experiment.

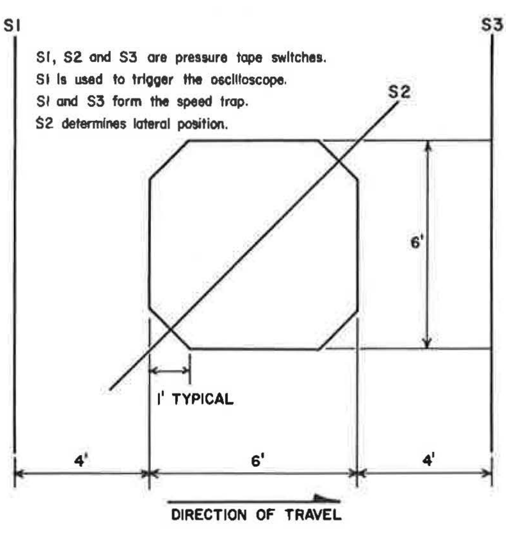
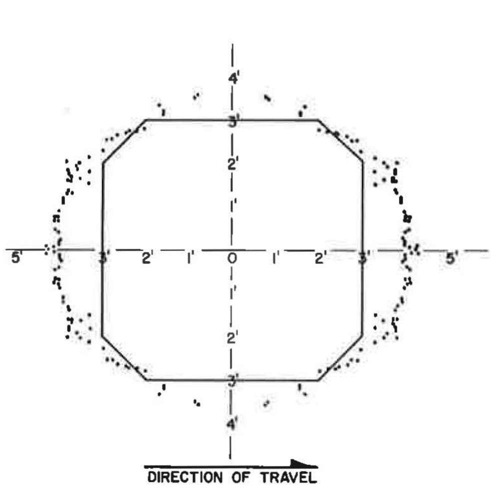


Figure 4. Effective loop shape.



4 at the 1 percent level of significance. It is rejected for all other estimators. This result agrees with the previous analysis that showed that estimators 1, 2, and 3 were not unbiased. Estimate 4 would be completely unbiased if the assumptions about the velocity and length distributions were met.

For the effectiveness of the alternate estimators in reducing the variance of the errors, the null hypothesis was tested. This tested the assumption that there are no differences between the variance obtained for each estimator. The comparisons were made with estimator 1. The null hypothesis was made at the 5 and 1 percent levels of significance by using the F-test. The hypothesis of no significant differences in variance between estimators is rejected for all estimators at both the 5 and 1 percent levels of significance. This means that estimators 2, 3, and 4 are all more efficient estimators than estimator 1.

ACKNOWLEDGMENT

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Appendix

DESCRIPTION OF EXPERIMENTS

The first experiments determined the effective shape of the loop magnetic field. The dependence on speed and the effect of changing detectors were investigated.

A schematic diagram of the experiment is shown in Figure 3. A magnetic loop and 2 pressure tape switches (S1 and S2) were located as shown. The pressure switches were connected with a battery and resistor such that a difference in voltage levels could be observed on a 4-trace oscilloscope whenever the switch was closed. The electronic detector output indicating the presence or absence of a vehicle in the field of the loop was also displayed. The oscilloscope was triggered by the front wheels of the vehicle making contact with the first tape switch, and a Polaroid picture was taken of the traces of the switch and detector outputs. From the photograph, the time occurrence of the relevant events was determined. Knowing the wheelbase of the car and the time required for the car to travel a distance equal to its wheelbase length, we determined the speed. Knowing the vehicle width and velocity, we calculated the position of the vehicle relative to the center of the loop.

For vehicles of known dimension, typical results are shown in Figure 4. The solid line denotes the geometric shape of the loop. The actual (bumper-to-bumper) car length is subtracted from the effective length to allow comparison of the curves for different vehicles. Figure 4 has the significance that, if any part of the vehicle enters the region enclosed by the dotted line, the detector is activated.

The experiment was designed so that actual speeds were determined to within ±0.4 mph and lane position to within ±3 in.

The loop was found to give symmetric results; hence, each data point is shown 4 times in Figure 4. No dependence on vehicle speed between 10 and 60 mph was detected.

The use of 3 different detectors for the same loop resulted in a 4 percent range for the total effective length L. The effective length L varies by less than 2 percent as long as a vehicle remains completely within the 12-ft lane. Comparisons of 4 different vehicles resulted in a range for the effective length of the loop alone of 1.8 ft.

Subsequent experiments are being performed to determine the distribution of effective lengths of vehicles actually using a freeway. The experiment is the same except that an additional pressure tape S3 is utilized to determine the wheelbase length for each vehicle. Two additional loops are located downstream with a 6-ft separation between loops in order to ascertain the difference in operating characteristics of 3 different loop-detector pairs. A 7-channel instrumentation tape recorder is used to record all events. The data are read and processed by a hybrid computer.

SAFETY EVALUATION OF FORCED WEAVING AS A TRAFFIC CONTROL MEASURE IN FREEWAY MAINTENANCE OPERATIONS

Kenneth A. Brewer, Department of Civil Engineering, Iowa State University

Effective traffic speed control during maintenance or reconstruction activity implies not only that the speed will be reduced enough to lower the conflict between work activity and traffic flow but also that no adverse conflicts will be introduced within the traffic stream. Speed control effectiveness and safety were analyzed for 2 examples of a traffic sign-traffic cone-traffic barricade pattern developed by the Iowa State Highway Commission. The pattern forces all through traffic to negotiate a single lane in a weaving pattern. The switchback from 1 lane to another is designed to control speeds. Results indicate that the weaving was highly effective when construction activity was visible; more than 50 percent of all vehicles sampled traveled below the posted temporary speed limit. When no construction activity was evident or when no forced weaving was used, fewer than 20 percent of all vehicles sampled complied with the posted speed limit. Analysis of the lane changing ahead of the weave pattern indicated that no excessive driver confusion was introduced in the approach to this unique speed-control pattern. Only 3 vehicles were observed to perform hazardous or unusual maneuvers in negotiating the advance-warning area.

•PREVIOUS in-depth studies of the anticipated impact of maintaining a completed 42,500-mile Interstate Highway System revealed some startling information (1, 2): (a) By 1975 it is estimated that the annual cost of maintaining shoulders and pavement will be 45 percent of the total Interstate System maintenance cost; and (b) there is little uniformity among the states in the control of freeway traffic when maintenance is performed.

As the Interstate System nears completion, we can reasonably expect the rate of use (extent of travel and traffic volumes) to rise even more rapidly than it has in the past when only parts and segments were completed. When maintenance work crews and freeway traffic interfere with each other, 2 courses of action are possible: (a) Traffic must detour around the work site on an alternate route or special detour roadway; and (b) traffic movement through the work site must be rigidly controlled for safety and efficiency.

It may be feasible to detour traffic onto alternate routes while only portions of the Interstate System are complete. Most of the segments begin and end at tie-in points to the existing primary system. However, when a considerable length of the freeway is completed, the spacing of interchanges limits re-entry points to the freeway. Also, no other type of facility has the traffic capacity that the freeway has or can provide the traffic speed. When the costs of special detour construction, the costs of increased congestion on primary routes, the effects of excessive traffic on alternate route maintenance, and the added costs of increased travel time are accumulated, the advantages of detouring freeway traffic outweigh the disadvantages in only limited situations.

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Because detouring freeway traffic is desirable only in restricted maintenance situations, the next alternative is to move the freeway traffic through the work zone. This choice eliminates the disadvantages of additional congestion, travel, and delay on alternate routes of the primary system. However, there are disadvantages to carrying traffic through a work site:

1. Delays are experienced by motorists in the vicinity of the work site;
2. Maintenance and reconstruction costs may be higher because traffic interferes with or restricts work crews; and
3. Hazards are added to both through traffic travel and work crew activity.

To minimize the hazards to both traffic and work crews requires that freeway traffic control be effective and efficient. An advance-warning signing system provides 2 characteristics of operation: positive speed control and smooth merging of traffic from closed lanes into open lanes.

Positive speed control brings high-speed freeway traffic down to a speed that does not unduly delay the through traffic yet is sufficiently slow that the work forces are not endangered by high-speed vehicles. No universal guidelines exist defining an appropriate speed for freeway traffic moving through a maintenance area. However, a survey of agencies responsible for freeway maintenance indicated that they attempted to limit traffic to 20 to 45 mph as a general rule. Whether the freeway traffic is limited to 30 or 40 mph is not so important from the workman's view as whether the control is effective. If all vehicles are moving at about 35 mph, workmen could adjust for the hazard of the traffic in their operation; but if some vehicles are traveling at 25 mph and some at 55 mph, the potential for a workman to misjudge the danger of a traffic hazard is greater. Thus, positive speed control is not only getting traffic to conform approximately to a posted speed limit (mandatory or advisory) through the work site but also minimizing the upper extreme speed deviations.

Smooth merging is fundamental to safe and efficient operations on a freeway any time 2 or more traffic streams are joined into one (3). Closing a through lane denies drivers the visual guidance of the gradually changing geometry of the pavement surfaces available at entrance ramps and main-lane drops. The configuration of signs, barricades, and traffic cones must provide sufficient guidance so that drivers can decide in which lanes they should be and carry out their decisions without abnormal or hazardous movements. Some highway agencies responsible for freeway maintenance have developed standard layouts for signs, barricades, and traffic cones that indicate rates of taper for merging traffic as a function of traffic speed or roadway geometry (2, 4, 5).

STUDY OBJECTIVE

The Iowa State Highway Commission (ISHC) uses a unique pattern of traffic cones and barricades to control traffic speeds on freeways when traffic is carried through a work site. The standard configuration is shown in Figure 1. This pattern is modified as required by roadway conditions at the work site. Usual modifications include changes in the spacing between consecutive signs, altering rates of taper, or spacing of the "lane switchback" because an entrance ramp or exit ramp is adjacent to the work site (or a similar constraint exists). The ISHC weaving section is a unique feature to slow freeway traffic and is, therefore, frequently an unfamiliar experience to drivers passing through Iowa on the Interstate Highway System. Potentially, this nonuniform traffic control pattern (with respect to states other than Iowa) could represent a special driving problem on the Iowa Interstate routes during the maintenance season. The National Science Foundation, therefore, funded a research initiation grant with the following objectives: (a) to estimate speed characteristics of vehicles traversing the weave section to indicate conformance with posted speed limits and to evaluate general safety of speeds; and (b) to evaluate merging and weaving of vehicles in advance of the weaving section.

DATA COLLECTION METHODOLOGY

Two classes of data were collected by 2 separate methods. Data on the weaving and merging behavior of vehicles as the traffic stream entered the advance-warning sign

Figure 1. Lane-closure pattern used in Iowa to slow traffic.

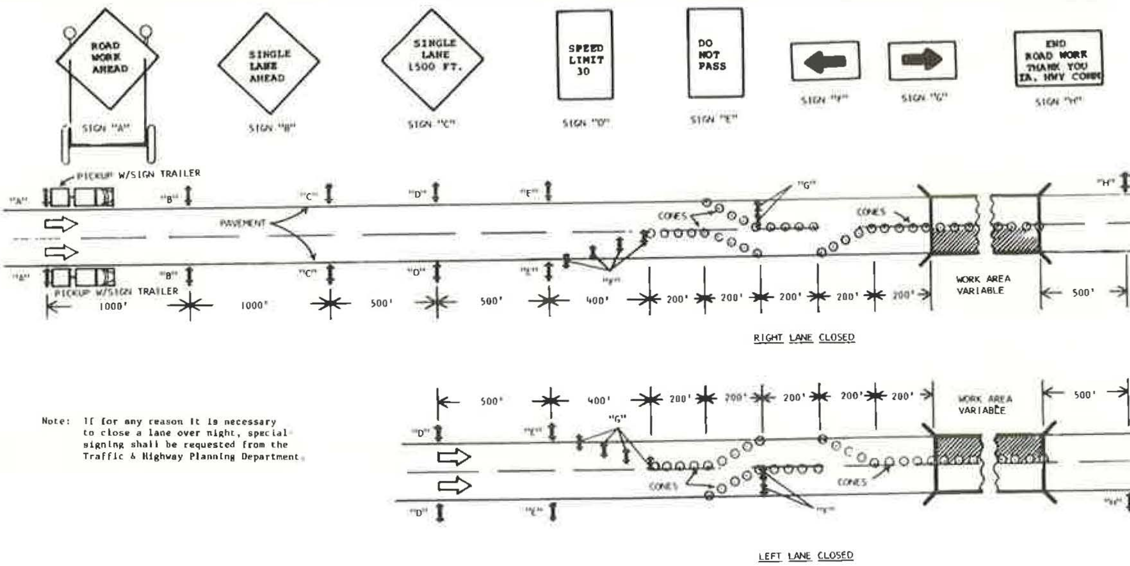
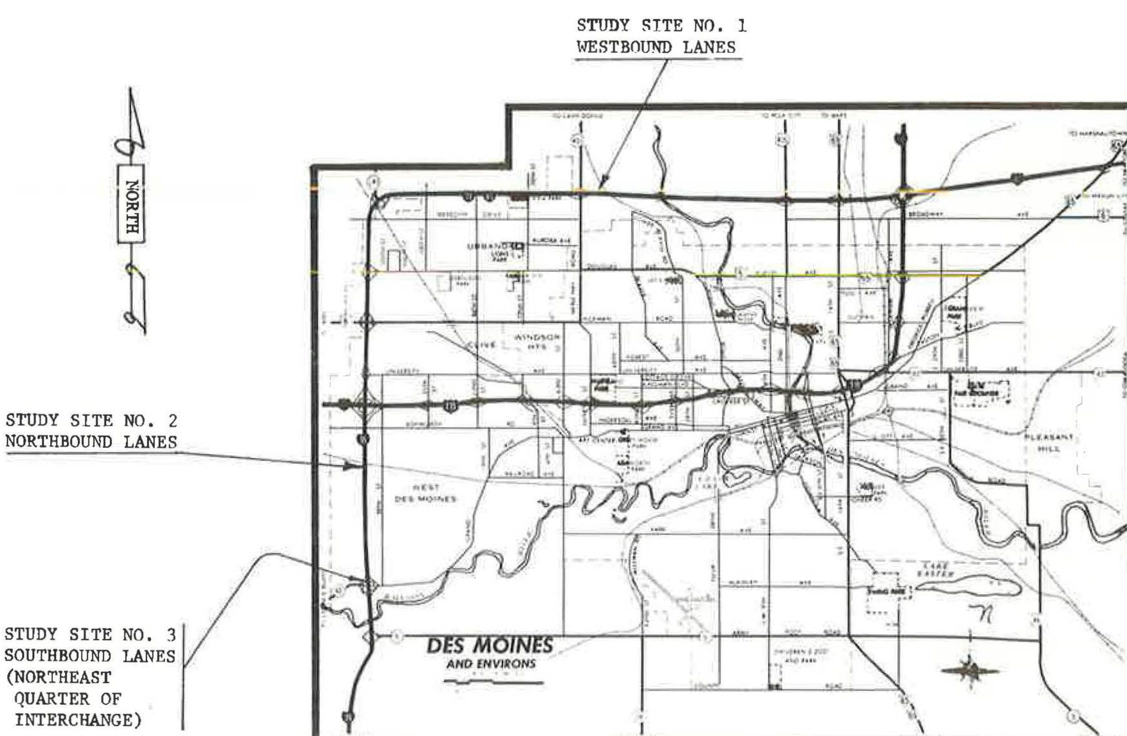


Figure 2. Site locations on I-35 and I-80.



area and maneuvered into the speed control weave section were collected with a 16-mm Beaulieu R-16 camera. The weaving data were collected on black and white film at 2 frames/sec. The Beaulieu R-16 holds a 100-ft roll of film so that each loading of the camera provided about 35 min of data. Camera observation positions were selected to look down on the traffic passing through the sign-barricade system, so the signs and barricades themselves were used as reference marks in retrieving data from the film. While the camera was operating, observers recorded the lane distribution of vehicles before the drivers were close enough to the signing system to react to sign information. This established a null condition reference from which to evaluate the weaving and merging as drivers progressed down the roadway. In addition, while the film was being exposed, a sample of vehicle license plates was recorded to classify the drivers as local (same county as study site), Iowa-other-than-local (vehicle registered in some Iowa county not containing the study site), or out-of-state. All trucks with multiple-state registration were classified as out-of-state. All government vehicles with an Iowa license plate not identifying a county were considered as Iowa-other-than-local vehicles. This classification may be a bit arbitrary and can possibly lead to some bias, but we felt those classes were as logical as could be devised to estimate the proportion of the drivers who might be unfamiliar with the traffic control scheme (out-of-state), those who might be unfamiliar with the control configuration at the study site (Iowa-other-than-local), and those who may be repeat drivers (local). In the early phases of the study methodology experimentation, we hoped that each vehicle observed on film could be positively identified by license plate; that proved to be impossible, so the gross sample identification was accepted.

The speed of vehicles after they had passed through the slow-down weave of traffic cones and barricades could not be obtained from the time-lapse photography. The close spacing of signs and cones in this area obscured the reference mark pattern, and the smaller vehicles were sometimes hidden from view. Enoscopes (flash boxes) were set up to measure the speeds of vehicles after the drivers had been slowed down by the weave section. The flash boxes were placed behind barricades and were hidden from the drivers' view. This limited the "gawker" effect of drivers' watching the collection of traffic data.

Maintenance work and reconstruction on Interstates 80 and 35 in the Des Moines area provided an opportunity to sample the traffic characteristics approaching this weave traffic control scheme. Figure 2 shows the location of 3 study sites.

1. At the Merle Hay Road on the north edge of Des Moines, the westbound traffic approaching the diamond interchange was observed. Figure 3 shows the location of signs, the camera location, and the roadway profile.
2. At the Minnesota and St. Louis Railroad on the southwest edge of the Des Moines area, northbound traffic was observed. No interchanges were near to provide disruptive influences. Figure 4 shows the layout of this site and the profile of the approach.
3. At the Iowa-90 interchange southwest of Des Moines, a temporary traffic control scheme was set up to direct traffic around a bridge joint-sealing operation. Instead of the full weave, only signs and a merge-left cone pattern were used to control through traffic. Speed data were collected here to compare to the speed control of the weave pattern. Figure 5 shows the layout at this site.

DATA ANALYSIS AND INTERPRETATION

Speed Control Data

The effectiveness of a speed control can be evaluated in several ways. One measure against which the method control can be evaluated is to calculate the proportion of the vehicles moving at or below the posted speed limit. At all 3 study sites, the speed limit was posted at 30 mph. At sites 1 and 3 vehicles were in rather poor compliance with the 30-mph speed limit, while at site 2 the majority of the vehicles (58.3 percent) were traveling at or below the speed limit after negotiating the weave section designed to

Figure 3. Plan and profile of site 1.

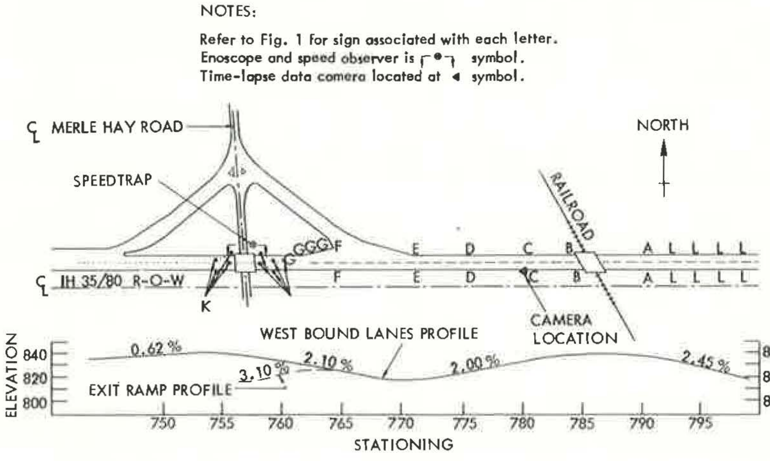


Figure 4. Plan and profile of site 2.

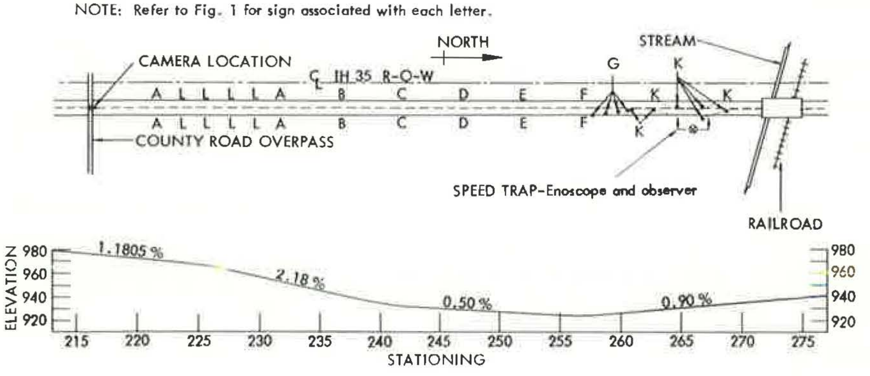
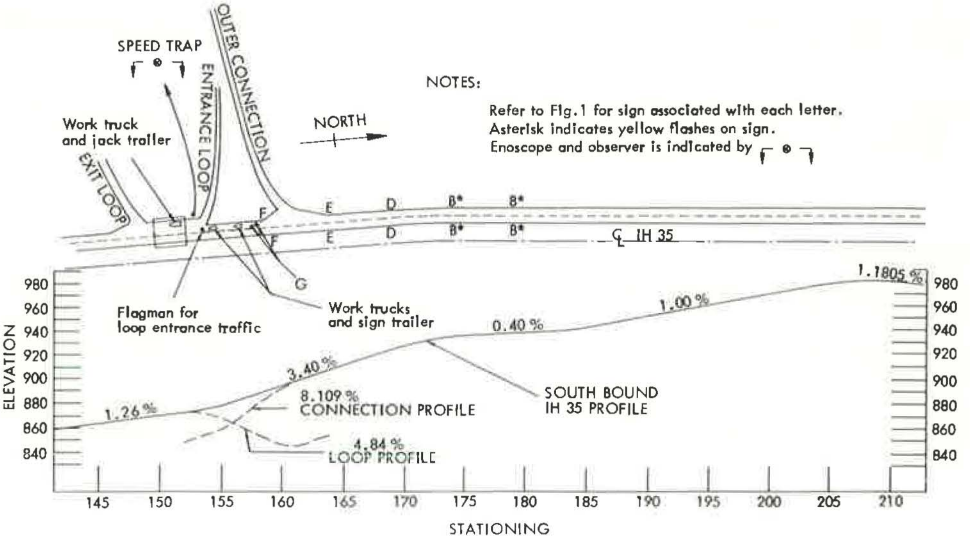


Figure 5. Plan and profile of site 3.



control speeds (Table 1). We assumed that the 30-mph speed was in fact the desirable speed to which vehicles should be limited. No attempt was made to establish what was the upper limit of a "safe" speed for traffic moving through the work site. Based on the percentage of vehicles obeying the posted speed limit, the conditions at site 2 produced a much safer operation.

If a spot speed sample is arrayed in order of increasing speed, frequently the dispersion pattern about the mean speed is very close to a normal distribution. Two published works have directed some effort to relating this "normality" of speeds to the general safety of travel (6, 7). The essence of both of these reports is that, if driving environment, traffic conditions, and driver desires combine to produce a normal distribution pattern of speeds, then speed has a minimal effect in traffic hazard (accident rate should be low). Further, if any condition causes the speeds to be non-normal in distribution, the skewness of the distribution is important in determining whether the speed characteristics are leading to unsafe operations. If most drivers are driving slower than the mean and a few drivers are traveling a great deal faster than the mean, a distribution would have a negative (left) skew. If most drivers are traveling faster and a few cautious persons are driving slower than the mean, the distribution would have a positive (right) skew. Skewness is shown in Figure 6. A study by the Ohio Department of Highways indicated that signing for safe speed limits reduced accidents and improved safety only when the speeds were skewed negatively or to the left (8). Further, Taylor found that the safest traffic operation was produced when speeds were normally distributed (6).

Data analysis results given in Table 1 indicate that speeds at all study sites were approximately normally distributed. The Kolmogorov-Smirnov goodness-of-fit test was applied to the speed samples, and the highest significance level was 0.10 for one of the speed samples at site 1. Common practice would not reject the hypothesis that the sample came from a normal distribution unless the significance level was 0.05 or 0.01. We would generally interpret this result to mean that all 4 samples were probably from normal distributions but that the data at site 1 are approaching non-normality. This test still is no basis for concluding that traffic operations at any of the sites were hazardous.

When the spot speed data were examined for skewness, however, one sample was significantly skewed. The skew test t-value was positive for all 4 samples, so at least no dangerous negative skew was observed (Table 1). Sample 1 taken at site 1 is significantly skewed, and the probability of a greater skewness test statistic, if the speeds were in fact not skewed, was 0.004. The results of this test indicate that, although the speeds contained in sample 1 have an undesirable skew, none of the samples has a potentially hazardous negative skew.

One other possible measure of safety is the maximum speed. Because all sites had the same posted speed limit of 30 mph, a lower maximum observed speed would potentially indicate a safer operation. Table 1 gives maximum speeds of 55, 42, and 61 mph for sites 1, 2, and 3 respectively.

The data analysis results suggest that a difference exists among the spot speed samples. The samples were compared on a pair-wise basis to test 2 hypotheses:

1. Were the variances (variability) of the 2 compared samples equal? and
2. Were the mean speeds of the 2 compared samples equal?

Table 2 gives the results of the pair-wise tests. Using the 0.05 level of significance as the rejection limit revealed that the variance for sample 4 was significantly different from those for samples 1, 2, and 3. The mean speeds of samples 1 and 2 were the only ones for which the hypothesis of equality could be accepted.

If the results of the speed data analysis are all combined in a qualitative judgment, the operation at site 2 was superior to the traffic operation at sites 1 and 3. The superiority of traffic operations through site 2 over site 3 could be explained as the superiority of the ISHC weave traffic control pattern over simply merging traffic out of a closed lane and relying on signs to control speeds. Any attempt to explain the superiority of the traffic operation at site 2 over site 1 is more complex. At site 1 the off-ramp may have been an influence contributing to some delay or abnormal operation.

Table 1. Spot speed data analysis.

Sample	Site	Sample Size	Mean Speed	Standard Deviation	Vehicles Traveling ≤30 mph (percent)	Test for Skewness: Calculated t	Test for Skewness: Significance Level	Test for Normality: Largest Deviation	Test for Normality: Significance	Speed Range (mph): Maximum	Speed Range (mph): Minimum
1	1	103	35.7	6.28	13.6	2.86	0.004	0.123	0.10	55.2	20.3
2	1	106	34.7	6.34	18.9	1.15	0.25	0.106	0.20	55.2	20.6
3	2	108	29.6	5.92	58.3	1.41	0.15	0.055	>0.20	42.8	18.6
4	3	107	39.1	8.14	13.1	1.18	0.25	0.069	>0.20	61.0	21.9

Figure 6. Relation of skew to sample mean value.

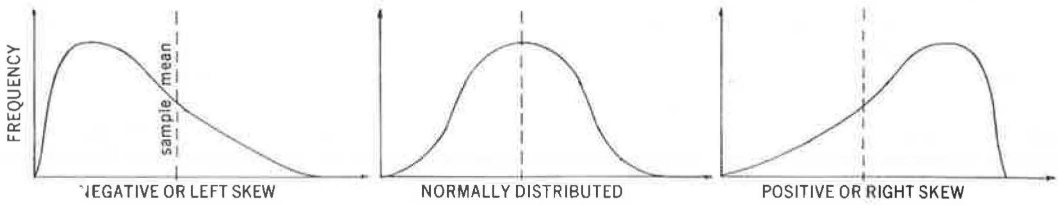


Table 2. Comparison of spot speed samples for equal variances and equal means.

Samples Compared	Calculated F	Significance Level	Equal Variances Hypothesis	Calculated t	Table t	Equal Means Hypothesis[a]
4-1	1.676	0.05	Rejected	3.4	1.984	Rejected
4-2	1.648	0.05	Rejected	4.4	1.984	Rejected
4-3	1.889	0.01	Rejected	9.78	1.984	Rejected
3-2	1.145	—[b]	Accepted	6.06	1.972	Rejected
3-1	1.127	—[b]	Accepted	7.26	1.972	Rejected
2-1	1.017	—[b]	Accepted	1.146	1.972	Accepted

[a]0.05 significance level. [b]Not significant.

Table 3. Percentage of vehicles in lanes at each distance reference point.

Site	Sample	DRP1 L1	DRP1 L2	DRP2 L1	DRP2 L2	DRP3 L1	DRP3 L2	DRP4 L1	DRP4 L2	DRP5 L1	DRP5 L2	DRP6 L1	DRP6 L2	DRP7 L1	DRP7 L2
1	1	0	100	09	91	53	47	63	37	70	30				
1	2	0	100	18	82	60	40	71	29	79	21				
1	3	0	100	19	81	56	44	64	36	71	29				
1	4	0	100	22	78	54	45	69	31	74	26				
1	—[a]	0	100	27	73	56	44	67	33	73	27				
2	1	0	100	08	92	40	60	60	40	68	32	75	25	84	16

Note: DRP = distance reference point; and L = lane.

[a]Combined.

Table 4. Lane volume at site 2 based on lane distribution at site 1.

Distance Reference Point	Light Volume Lane	Expected Limit: Lower	Expected Limit: Upper	Observed	Distance Reference Point	Light Volume Lane	Expected Limit: Lower	Expected Limit: Upper	Observed
1	1	0	0	0	4	2	37	61	58
2	1	28	51	12[a]	5	2	28	51	46
3	2	51	77	88[a]					

[a]Observed value at site 2 falls outside the range of values expected 95 percent of the time based on the traffic pattern of lane placement at site 1.

If the off-ramp was an influence, it would seem to have delayed the traffic and slowed speeds rather than to have increased speeds. In the author's opinion, the main factor was the presence, in full view of the driver, of heavy equipment and construction activity on the roadway at site 2. At site 1 no construction or maintenance activity was visible on the through roadway to stimulate the drivers' desire for safety. This is a topical area of freeway maintenance research for further study.

Weaving and Merging During Advance Warning

Time-lapse photography data on the weaving between traffic streams and merging-diverging manuevers of traffic streams were analyzed in order to gain further insight into the differences in traffic operations between site 1 and site 2. The camera was positioned as far away from the point where 1 of the 2 lanes was closed as was physically possible. The constraint on camera location at both site 1 and site 2 was the farthest vantage point from which a full view of both the vehicles and the roadway could be maintained (Figs. 3 and 4). At site 1 the camera was about 2,000 ft from the point where lane 1 (outside lane) was closed completely. The camera at site 2 was about 1.1 miles from the point of closure on lane 1.

The lane distribution of vehicles approaching the weave speed control section is given in Table 3. Distance reference point 1 is the point where the traffic cones completely blocked the curb lane, reference point 2 is at the beginning of the traffic cone taper, and reference points 3, 4, and so on are at the locations of the major signs successively more distant from the beginning of the taper. Four film samples were available from site 1, and 1 sample was available from site 2. Data sampling is not so unbalanced as it might seem superficially. Site 1, which has an exit ramp, sharp changes in vertical alignment, and a limited distance over which unfamiliar drivers had full view of the temporary traffic control system, could have been expected to produce large variations in lane placement. The data collected and analyzed, however, yielded quite consistent lane-distribution percentages as given in Table 3.

It was of interest to estimate what the effect of the exit ramp is on traffic. At site 1 the exit ramp traffic was 34 percent of the total volume, so it could have been a major influence causing the differences in lane placement. The lane distribution at each reference point at site 1 was taken as the reference basis for a binomial probability expectation of the number of vehicles to be found in the light volume lane at site 2. If the traffic were similar in behavior (that is, the exit ramp has no effect on lane distribution), the expected range of vehicles predicted at site 2 based on site 1 data should include the actual observed vehicle count. To calculate the expected range, we used the normal distribution approximation to the binomial and chose a 0.05 significance level. The results are given in Table 4. At reference points 2 and 3 the lane distribution at site 1 fails to predict the number of vehicles at the corresponding point at site 2. Reference point 3 is at the beginning of the exit ramp deceleration lane taper, and reference point 2 is 650 ft farther (just beyond the exit ramp gore for site 2). Reference point 4 is 500 ft upstream from reference point 3. These results can be interpreted to mean that the effect of the percentage of the exiting traffic only extended for about 500 ft each side of the exit ramp itself.

At each site the proportion of traffic that was local in nature might be a factor in the degree to which the exit ramp traffic influenced lane placement. At site 1 samples of license plates indicated that during the studies 27 percent of the vehicles were registered in the local county, 15 percent were registered in other Iowa counties, and 58 percent were out-of-state vehicles. At site 2 the same categories were 28, 42, and 30 percent respectively. Although the relative proportion of vehicles that were out of state and Iowa but not local were quite different at the 2 sites, the percentage of local vehicles was nearly identical. It does not appear that vehicles representing repeat drivers through the site are significant enough to bias the findings. If no temporary traffic controls were in effect, the high percentage of exiting traffic should encourage the through traffic to move left into lane 2 in the vicinity of the exit ramp. However, far more traffic stayed in the outside lane and merged left as it approached the traffic cone taper than did so at site 2 where no exit ramp influenced the traffic (Table 4).

If the exit ramp traffic at site 1 confused the through drivers, or if the through drivers in lane 1 constrained the exiting traffic, then some hazardous or unusual traffic maneuvers in the vicinity of the exit ramp should be observed. A total of 1,636 consecutive vehicles were analyzed in detail for lane changing and shifting of positions within the traffic stream as it moved through the exit ramp area. Of these, 2 vehicles exited from the median lane in front of a platoon in the right lane and 1 vehicle stopped in the curb lane at the start of the traffic cone taper. This indicates that the communication with the driver is at least adequate. Drivers apparently can perceive what is required of them to negotiate the speed control weave without unduly interfering with the traffic, which is more concerned with the permanent road signs for exits and so on.

GENERAL CONCLUSIONS

Based on the somewhat limited data provided by a research initiation project, the control of freeway traffic through a maintenance or reconstruction area by the use of the Iowa State Highway Commission weave pattern is a safe and effective operation. Speeds through the areas where work is in progress are controlled significantly better with the use of the ISHC pattern than with the use of signs alone. Driver communication is adequate for safe lane changing under the present light-to-moderate volumes experienced on Iowa Interstate routes.

FURTHER QUESTIONS

As in any new research, more questions were raised in this study than were answered. The following are some of these.

1. How far back from the lane closure do drivers react to the warning signs?
2. Are there more effective communication means that can reduce the amount of signing and thereby reduce the cost of control?
3. What is the capacity of the weave speed control section for various rates of taper within the weave section?
4. What are the ranges of traffic volumes over which this weave section is effective? How is its volume-capacity ratio related to its effectiveness?
5. How significant is platoon behavior in communicating the proper movement through the control section to following drivers?

Perhaps some of these questions can be answered in the future. Before general application of this unique control scheme is attempted, it would be helpful to be able to predict its usefulness, effectiveness, and relative safety.

ACKNOWLEDGMENTS

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