Feasibility of an Exclusive Lane for Buses on the San Francisco-Oakland Bay Bridge

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The study was initiated by a request from the U. S. Bureau of Public Roads for an evaluation of the potential for reserving one or more exclusive bus lanes on the San Francisco-Oakland Bay Bridge. This section of highway has the greatest peak-hour bus concentration in California, and delay occurs because demand exceeds capacity during peak periods. The existing traffic conditions were surveyed for both morning and evening peak periods. The data obtained included capacity of bridge, number of persons using each mode, volumes (automobiles and buses), travel times, and demand. This information was used to determine the present person-delay being suffered. These data were then used in a simulation of conditions with an exclusive bus lane in effect. The assumption was made that the lane used exclusively for buses was previously a lane used for mixed traffic in the same direction. A graph was developed that showed person-delay with and without an exclusive bus lane as a function of modal split. This paper concludes that an exclusive bus lane on the Bay Bridge is not feasible because the increased delay to automobile users would far exceed the savings to the bus passengers. At the present demand there is no modal split that would make an exclusive bus lane feasible. The conclusions were based principally on recurrent congestion. If nonrecurrent congestion such as that caused from breakdowns and accidents had been included in the delay, the exclusive bus lane alternative would have been even more detrimental.

This study was initiated when the Bureau of Public Roads requested that an evaluation be made of the potential for reserving one or more exclusive bus lanes on the San Francisco-Oakland Bay Bridge (SFBB). The position of the Federal Highway Administration on the reservation of freeway lanes for buses is stated in the Bureau's Instructional Memorandum 21-13-67. The SFBB is a 10-lane facility with 5 westbound lanes on the upper deck and 5 eastbound lanes on the lower deck. The bridge has no shoulders, and when accidents or stalls occur the capacity of the bridge is reduced by at least 1 lane. The area in which the bridge is located is shown in Figure 1.

Data for this report were obtained from several sources. The capacity of the bridge was determined from mainline counts taken near the tunnel on Yerba Buena Island by the Division of Bay Toll Crossings and was verified by counts and aerial photographs of queuing at the east end of the bridge. Counts of volumes on the ramps feeding the bridge were obtained from the San Francisco office of the Division of Highways. Bridge data, including a very comprehensive bus travel-time study during peak periods, were also furnished by the Division of Bay Toll Crossings. Other data, including bus counts and occupancy, were obtained from semi-annual traffic surveys conducted by the University of California.
PRESENT CONDITIONS

The present volume and composition of traffic during the peak congested periods are given in Table 1. The present average off-peak travel time for buses in the westbound direction from the Bay Bridge Toll Plaza to the San Francisco Terminal Building is 8.25 min and the travel time in the eastbound direction from terminal building to toll plaza is 8.75 min. Details on travel time and delay for buses are given in the Appendix. Eastbound, off-peak travel takes 30 sec longer than the westbound, off-peak travel because the buses are starting from almost a complete stop on an approximately 3 percent grade.

During the peak period under ideal conditions (that is, good weather, daylight, and no lane blockage), the average travel time for buses in the westbound direction was approximately 11 min and in the eastbound direction, 11.3 min.

The maximum delay to buses leaving the toll plaza in the westbound direction occurred between 7:35 and 7:45 a.m. This is the period when queuing from the merge section is greatest. The maximum delay obtained from bus travel times on 9 different weekdays amounted to 4.75 min. Most of this delay is incurred in the merge section between the bridge and the toll plaza.

The delay for buses in the eastbound direction is fairly constant throughout the latter part of the peak period (5:00 to 6:00 p.m.). This is because the majority of the delay being experienced is running delay on the bridge. (Running delay is the delay caused by the natural reduction in speed as flow rates increase, and queuing delay is that caused by demand exceeding capacity.) Variability is observed only when accidents or stalls

### Table 1

<table>
<thead>
<tr>
<th>Time</th>
<th>Vehicles</th>
<th>People</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Buses</td>
<td>Automobiles</td>
</tr>
<tr>
<td></td>
<td>Number</td>
<td>Percent</td>
</tr>
<tr>
<td>Peak hour&lt;sup&gt;b&lt;/sup&gt;</td>
<td>7:10 to 8:10 a.m.</td>
<td>340</td>
</tr>
<tr>
<td>Peak period</td>
<td>6:35 to 6:25 a.m.</td>
<td>433</td>
</tr>
<tr>
<td>Peak hour&lt;sup&gt;b&lt;/sup&gt;</td>
<td>4:35 to 5:35 p.m.</td>
<td>303</td>
</tr>
<tr>
<td>Peak period</td>
<td>4:10 to 5:45 p.m.</td>
<td>374</td>
</tr>
</tbody>
</table>

<sup>a</sup>Includes trucks.  
<sup>b</sup>Hour of maximum flow of people across the bridge and not necessarily hour of maximum flow of vehicles.
occur on the bridge. On one of the data collection days 2 separate lane blocks in the eastbound direction lasted a total of 27 min and caused maximum delays of 8.5 min per bus. Although nonrecurrent congestion occurs frequently and must be anticipated, it is not included in this analysis because of its extreme day-to-day variability.

At the present time the buses are suffering less delay than the automobiles during the peak congested periods. This is true in both directions. In the eastbound direction the buses presently have an exclusive entrance to the bridge, and the delay they incur in getting onto the bridge is negligible. The automobiles, on the other hand, queue up on every ramp leading to the bridge.

The westbound direction generally has a continuous flow except at the merging section near the beginning of the bridge. Six lanes feed into the toll plaza area, but only 5 lanes are on the bridge (Fig. 2). Because the capacity on the bridge is less than the capacity at the toll booths, queuing occurs between the bridge and the toll booths. Even though it is not mandatory, the buses predominantly use the right lanes in the 17-lane toll booth area because the drivers have found from experience that operation is smoother in the right lanes where the traffic volumes are lighter. The queuing delay being experienced between the toll booths and the bridge, a distance of 3,750 ft, is less for the buses than for the automobiles. The average queuing delay per automobile during the morning peak on October 9, 1968, was 2.3 min, whereas the average queuing delay per bus was only 1.5 min. The maximum queuing delay for automobiles was 4.3 min and for buses only 2.75 min.

The average delay on the bridge (running delay) for automobiles is approximately the same as that for buses during the peak periods. The running delay is the difference between low-volume speeds (43 mph for buses and 45 to 50 mph for automobiles) and capacity-volume speeds (37 mph).

Because of the difficulty in obtaining data on the many San Francisco approaches, the queuing delay for automobiles was determined only for the westbound direction. The total queuing delay is much greater in the eastbound direction.

**CONDITIONS WITH AN EXCLUSIVE BUS LANE**

Only the westbound direction is considered in the analysis of the reservation of 1 of the existing 5 lanes for exclusive use by buses, leaving only 4 lanes available for vehicles other than buses. Details of the analysis are given in the Appendix; the results are discussed in the following.

If one lane is used exclusively by buses, the current travel demand and the number of people using each of the 2 modes will increase the total queuing delay in the westbound direction from 65,600 to 289,000 person-minutes even though the bus passengers suffer no delay. The total running delay will decrease from 36,900 to 36,800 person-minutes, and the total delay will increase from 102,500 to 325,800 person-minutes. The maximum queuing delay per vehicle will increase from 4.3 to 16 minutes, and the maximum number of vehicles in queue will increase from 620 to 1,950.

A queue of this magnitude will cause congestion in the East Bay Distribution Structure, which is the interchange among I-80, I-580, and Cal-17 (Fig. 3), and will adversely affect 3 major freeways and many motorists on these freeways who are destined for locations other than San Fran-

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Figure 2. San Francisco-Oakland Bay Bridge Toll Plaza (looking west).
Because of the excessive queuing that will occur in the distribution structure, the buses themselves will incur additional delay upstream from the toll plaza exceeding the savings that they gained by having an exclusive bus lane on the bridge. Congestion will be present for approximately 3 hours from 6:30 to 9:30 a.m.

Although not calculated, the delay caused by an exclusive bus lane in the eastbound direction will be much more severe because of lack of storage space for the queued vehicles. This in turn will create delays for large numbers of persons not destined for the bridge. Congestion will extend well beyond the Central Skyway connection to the James Lick Freeway. The bus traffic will save very little (1 to 2 min on the bridge) because even under the present conditions the buses are experiencing only 30 sec queuing delay.

In the event of accidents or stalls, the exclusive bus lane will be quite advantageous to the buses, unless the lane blockage occurs in the exclusive bus lane, and the savings to bus passengers may seem quite large when compared with the delay suffered by motorists. The result, however, would be to greatly increase total person-delay above the calculated figures that are based on incident-free operation.

**EFFECT OF CHANGE IN MODAL SPLIT**

Figure 4 shows person-minutes of delay for various percentages of people riding buses with and without an exclusive bus lane and the current demand that stays constant regardless of the modal split. The present peak-hour demand is 25,200 persons. The figure is based on the following stipulations:

1. Five lanes are available, either 5 mixed or 4 for automobiles and trucks and 1 for buses.
2. Delay is incurred when queuing is present.
3. Automobile occupancy is 1.49 persons.
4. One bus is equivalent to 2 automobiles in mixed traffic stream.
5. Capacity of bridge with 5 lanes available for mixed traffic is 8,800 vehicles per hour.
6. Capacity of bridge for other vehicles with only 4 lanes available is 7,280 per hour.

The current modal split during the peak period is 44 percent for buses and 54 percent for automobiles; buses are much lower both before and after this period. Composition of demand for other modal splits, which caused the peak period to vary, was determined by allocating the changes made in proportion to the present demand.

As Figure 4 shows, if an exclusive bus lane is in effect, the delay will not be eliminated until nearly 57 percent of the people ride buses. However, if 57 percent rode buses and mixed traffic were allowed on all 5 lanes, there would be no delay for anybody and there would be considerably more freedom of movement in the traffic stream. In other words, the bridge would be operating at approximately 82 percent of capacity with mixed traffic in all 5 lanes, but at 100 percent of capacity with 1 lane for buses and 4 lanes for automobiles and trucks. It can be concluded that at the present demand no modal split would make an exclusive bus lane feasible.
Currently buses traveling eastbound are bypassing the major congested area, the approaches to the bridge. They are able to do this because they have an exclusive bus ramp to the bridge.

A study was made of the possibility of allowing buses to bypass the congestion westbound between the toll booths and the bridge. This could easily be done by reserving the far right toll booth exclusively for buses and striping an exclusive bus lane from this booth to near the beginning of the bridge where the buses would then merge with other traffic in the right lane. This would bring the buses in at the head of the line. The only delay the buses would incur would be the nominal running delay crossing the bridge.

The merits of this plan were determined by a detailed bus travel-time study to determine how long the buses are being delayed in this merging section between the toll booths and the bridge. It was discovered that the average delay to buses during the peak period is only 1.5 min compared to 2.3 min for all other vehicles. The bus drivers have discovered the advantage of using the right lanes even though they are not mandatory. Automobiles avoid the right lanes because of the high bus density. Therefore, the buses have an unofficial bypass. The major queuing delay being suffered is near the bridge where the buses merge into the mainstream of traffic. This problem would be present even if an exclusive bus bypass were in effect.

The bypass lane could not extend onto the bridge for this would mean eliminating a lane for use by other traffic. This would have the same effect on queuing delay as an exclusive bus lane across the entire bridge. At the present time an exclusive bus bypass between the toll booths and the bridge will result in a 1- to 1.5-min savings per bus. Some additional delay will be suffered by other vehicles at the merging section.
The savings to buses become quite insignificant when considered as part of the total trip time from point of origin to point of destination.

The eastbound approach is currently not a problem for the buses. Observations in this area revealed 3 detriments to vehicle traffic flow:

1. The exclusive bus entrance has an inadequate merging lane and is controlled by a yield sign;
2. The buses are starting from almost a complete stop on an approximately 3 percent grade; and
3. The buses begin changing lanes at slow speeds as soon as they enter the bridge.

In addition to helping buses, improvements in the merging geometrics so that the buses can begin the grade at a higher speed will improve the flow of traffic. After entering the bridge, the buses should stay in the same lane until reaching the summit of the west bay crossing.

**NONRECURRENT CONGESTION**

This analysis is based principally on recurrent congestion, congestion caused from inadequate capacity. Nonrecurrent congestion, such as that caused from breakdowns and accidents, has not been discussed.

Traffic surveys by the University of California of stalls and accidents on the bridge during peak periods on October 18 and 19, 1967, and April 16 and 17, 1968, show that an average of 1⅛ incidents occur daily in the westbound direction and last for an average of 19 min each. In the eastbound direction, approximately 5 incidents occur daily and last for an average of 12 min each. If nonrecurrent congestion had been included in the analysis, the exclusive bus lane alternative would have been even more detrimental.

**CONCLUSION**

An exclusive bus lane on the San Francisco-Oakland Bay Bridge is not feasible. The average delay currently on the bridge is only 1 to 2 min over a 5-mile section. The delay incurred in the merging section downstream from the toll plaza amounts to an average queuing delay of 1.5 min for buses and 2.3 min for other vehicles. An exclusive bus lane will increase delays to the automobiles, and the resulting loss will far exceed the savings to the bus traffic because available capacity in the bus lane will be wasted. The net increase in delay, if an exclusive bus lane is in effect, will be approximately 223,300 person-minutes daily during the westbound morning peak and a much larger amount during the eastbound evening peak.

Even though the exclusive bus lane will result in savings to buses while on the bridge, losses will be incurred upstream from the bridge. Other vehicles being delayed because of the reduced number of lanes for their use on the bridge will be backed up far upstream. The buses will suffer additional delay from this upstream congestion unless an exclusive bus lane extends back to the point where they enter the freeway. Because the buses now enter the freeway from many different ramps on 3 separate freeways, the provision of exclusive lanes from all of those origins is not feasible. None of this bus delay or delay to vehicles not destined for the bridge is included in the 223,300 person-minutes of increased delay per morning.

An exclusive bus bypass through the toll plaza area definitely has merit. Increased demands in the future may prove this to be a valuable alternative. At the present demand, no modal split would make an exclusive bus lane feasible.

These conclusions have been based principally on recurrent congestion. If nonrecurrent congestion, such as that caused from breakdowns and accidents, were included in the delay, the exclusive bus lane alternative would be even more detrimental.

**ACKNOWLEDGMENTS**

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project was undertaken in cooperation with the U. S. Department of Transportation, Federal Highway Administration, Bureau of Public Roads. The opinions, findings, and conclusions expressed in this paper are those of the Division of Highways and not necessarily those of the Bureau of Public Roads.

Appendix

BUS TRAVEL TIME AND DELAY

Present Conditions

Tabulations of travel times for buses between the San Francisco Terminal Building and the Bay Bridge Toll Plaza were made by the Division of Bay Toll Crossings for both the eastbound and westbound directions. Table 2 gives information for conditions with no nonrecurrent congestion extracted from these tabulations.

Figure 5 shows the total travel time for buses between the toll booths and the terminal in the westbound direction on October 9, 1968, during the morning peak period. No nonrecurrent congestion existed, and the weather was clear; conditions were very close to perfect. In the eastbound direction, the travel time on the bridge during the peak period is approximately 2 min greater than that during the off-peak period.

Figure 5. Bus travel times during peak period westbound from Toll Plaza to San Francisco Terminal.
**Table 2**

<table>
<thead>
<tr>
<th>Peak or Delay</th>
<th>Westbound</th>
<th>Eastbound</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average peak</td>
<td>10.85</td>
<td>11.30</td>
</tr>
<tr>
<td>Average off peak</td>
<td>8.25</td>
<td>8.75^{a}</td>
</tr>
<tr>
<td>Average delay</td>
<td>2.60</td>
<td>2.55</td>
</tr>
<tr>
<td>Maximum peak</td>
<td>13.00</td>
<td>13.00</td>
</tr>
<tr>
<td>Maximum delay</td>
<td>4.75</td>
<td>4.25</td>
</tr>
</tbody>
</table>

^{a}Eastbound off peak takes 30 sec longer than westbound off peak because buses are starting from almost a complete stop on an approximately 3 percent grade.

**Table 3**

<table>
<thead>
<tr>
<th>Time (a.m.)</th>
<th>Vehicles</th>
<th>Cumulative Vehicles</th>
<th>15-Min Flow Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>6:00</td>
<td>837</td>
<td>0</td>
<td>5,348</td>
</tr>
<tr>
<td>6:00</td>
<td>637</td>
<td>837</td>
<td>5,232</td>
</tr>
<tr>
<td>6:15</td>
<td>1,308</td>
<td>2,145</td>
<td>7,484</td>
</tr>
<tr>
<td>6:45</td>
<td>1,993</td>
<td>4,016</td>
<td>7,972</td>
</tr>
<tr>
<td>7:00</td>
<td>2,435</td>
<td>6,009</td>
<td>9,740</td>
</tr>
<tr>
<td>7:15</td>
<td>2,455</td>
<td>8,444</td>
<td>9,820</td>
</tr>
<tr>
<td>7:30</td>
<td>2,350</td>
<td>10,899</td>
<td>9,400</td>
</tr>
<tr>
<td>7:45</td>
<td>1,943</td>
<td>13,249</td>
<td>7,772</td>
</tr>
<tr>
<td>8:00</td>
<td>2,048</td>
<td>15,192</td>
<td>6,192</td>
</tr>
<tr>
<td>8:15</td>
<td>1,748</td>
<td>17,240</td>
<td>6,992</td>
</tr>
<tr>
<td>8:30</td>
<td>1,599</td>
<td>18,888</td>
<td>6,396</td>
</tr>
<tr>
<td>8:45</td>
<td>1,466</td>
<td>20,567</td>
<td>5,864</td>
</tr>
<tr>
<td>9:00</td>
<td>1,350</td>
<td>22,053</td>
<td>5,400</td>
</tr>
<tr>
<td>9:15</td>
<td>1,250</td>
<td>23,403</td>
<td>5,000</td>
</tr>
<tr>
<td>9:30</td>
<td>24,653</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Demand and Capacity Westbound**

Flow through the toll booths in the westbound direction was used as the demand. Table 3 gives the demand on October 9, 1968, from the hours of 6:00 to 9:30 a.m. The capacity of the toll booths is greater than the capacity of the bridge; therefore, queuing occurs between the bridge and the toll booths. The capacity of the bridge was determined from mainline counts taken near the tunnel on Yerba Buena Island on 4 different days. These counts were taken in both directions far downstream from
bottleneck sections. From these counts, the capacity for both directions was determined to be 8,800 vehicles per hour.

A reduced-scale representation of a cumulative plot of demand and capacity with respect to time is shown in Figure 6. The area within the demand and capacity curves is the total delay incurred in vehicle-minutes. The difference between the 2 curves on the vertical axis gives the number of vehicles in queue at any time. The difference between the 2 curves on the horizontal axis gives the delay per vehicle at any time.

The following information was obtained from Figure 6. In the bus queuing delay, 1.5 min is the average delay per bus during the morning peak period as determined from the bus travel-time survey.

<table>
<thead>
<tr>
<th>Period of delay</th>
<th>Total queuing delay, vehicle-minutes</th>
<th>Bus queuing delay (433 buses × 1.5 min), vehicle-minutes</th>
<th>Total automobile delay, vehicle-minutes</th>
<th>Maximum delay, min</th>
<th>Maximum number of vehicles in queue</th>
<th>Total number of automobiles delayed (13,200 - 433)</th>
<th>Mean automobile delay (29,350/12,767), min per automobile</th>
</tr>
</thead>
<tbody>
<tr>
<td>6:55 to 8:25 a.m.</td>
<td>30,000</td>
<td>650</td>
<td>29,350</td>
<td>4.3</td>
<td>620</td>
<td>12,767</td>
<td>2.3</td>
</tr>
</tbody>
</table>

**TABLE 4**

**DELAY INCURRED BY BUSES DURING PEAK PERIOD WESTBOUND ON OCTOBER 9, 1968**

<table>
<thead>
<tr>
<th>Time (a.m.)</th>
<th>Number of Buses</th>
<th>Occupancy per Bus</th>
<th>Queuing Delay Toll Plaza to Bridge</th>
<th>Running Delay East End of Bridge to S.F. Terminal</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Avg. Delay per Bus (min)</td>
<td>Total Delay (person-min)</td>
</tr>
<tr>
<td>6:55</td>
<td>14</td>
<td>32.2</td>
<td>0.25</td>
<td>117</td>
</tr>
<tr>
<td>7:00</td>
<td>11</td>
<td>42.4</td>
<td>0.25</td>
<td>117</td>
</tr>
<tr>
<td>7:05</td>
<td>18</td>
<td>42.4</td>
<td>0.25</td>
<td>191</td>
</tr>
<tr>
<td>7:10</td>
<td>25</td>
<td>42.4</td>
<td>0.50</td>
<td>531</td>
</tr>
<tr>
<td>7:15</td>
<td>21</td>
<td>42.4</td>
<td>0.75</td>
<td>667</td>
</tr>
<tr>
<td>7:20</td>
<td>30</td>
<td>42.4</td>
<td>1.25</td>
<td>1,590</td>
</tr>
<tr>
<td>7:25</td>
<td>23</td>
<td>42.4</td>
<td>1.75</td>
<td>1,705</td>
</tr>
<tr>
<td>7:30</td>
<td>28</td>
<td>35.3</td>
<td>2.00</td>
<td>1,975</td>
</tr>
<tr>
<td>7:35</td>
<td>38</td>
<td>35.3</td>
<td>2.75</td>
<td>3,680</td>
</tr>
<tr>
<td>7:40</td>
<td>39</td>
<td>35.3</td>
<td>2.75</td>
<td>3,780</td>
</tr>
<tr>
<td>7:45</td>
<td>26</td>
<td>35.3</td>
<td>2.25</td>
<td>2,065</td>
</tr>
<tr>
<td>7:50</td>
<td>29</td>
<td>35.3</td>
<td>2.00</td>
<td>2,050</td>
</tr>
<tr>
<td>7:55</td>
<td>26</td>
<td>35.3</td>
<td>1.75</td>
<td>1,610</td>
</tr>
<tr>
<td>8:00</td>
<td>34</td>
<td>22.3</td>
<td>1.25</td>
<td>760</td>
</tr>
<tr>
<td>8:05</td>
<td>21</td>
<td>22.3</td>
<td>1.25</td>
<td>588</td>
</tr>
<tr>
<td>8:10</td>
<td>25</td>
<td>22.3</td>
<td>0.75</td>
<td>420</td>
</tr>
<tr>
<td>8:15</td>
<td>13</td>
<td>22.3</td>
<td>0.25</td>
<td>73</td>
</tr>
<tr>
<td>8:20</td>
<td>12</td>
<td>22.3</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Total</td>
<td>433</td>
<td>—</td>
<td>21,919</td>
<td>—</td>
</tr>
<tr>
<td>Avg.</td>
<td>—</td>
<td>1.51</td>
<td>—</td>
<td>1.10</td>
</tr>
</tbody>
</table>
According to the University of California traffic survey, the average vehicle occupancy is 1.49 persons between the hours of 6:55 and 8:25 a.m. Therefore, queuing delays in person-minutes are

Automobile queuing delay \((20,350 \times 1.49)\) 43,700
Bus queuing delay (from Table 4) 21,900
Total queuing delay 65,600

The running delay incurred on the bridge was caused by high volumes. This delay extended from 6:55 to 8:25 a.m., the approximate time period of the queuing delay.

The average running delay is 1.1 min per vehicle during the congested period as determined by the bus travel-time survey on October 9, 1968. It is assumed that during the peak periods, running delay per automobile is the same as the running delay per bus. Total running delays in person-minutes between 6:55 and 8:25 a.m. were determined as follows:

Total people in 433 buses (from Table 4) 14,797
Total people in automobiles \((12,767 \times 1.49)\) 19,023
Running delay for buses (from Table 4) 15,950
Running delay for automobiles \((19,023 \times 1.1 \text{ min})\) 20,925
Total running delay 36,875
Total queuing delay 65,600
Total delay 102,475

Conditions With an Exclusive Bus Lane

There were 300 buses crossing the bridge during the peak hour. For the purpose of these calculations, we have assumed that in the traffic stream a bus is equivalent to

<table>
<thead>
<tr>
<th>Time (a.m.)</th>
<th>Total Vehicles</th>
<th>Minus Buses</th>
<th>Vehicles Without Buses</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Number</td>
</tr>
<tr>
<td>6:00</td>
<td>837</td>
<td>10</td>
<td>827</td>
</tr>
<tr>
<td>6:15</td>
<td>1,308</td>
<td>10</td>
<td>1,298</td>
</tr>
<tr>
<td>6:30</td>
<td>1,871</td>
<td>15</td>
<td>1,856</td>
</tr>
<tr>
<td>6:45</td>
<td>1,993</td>
<td>29</td>
<td>1,964</td>
</tr>
<tr>
<td>7:00</td>
<td>2,435</td>
<td>54</td>
<td>2,381</td>
</tr>
<tr>
<td>7:15</td>
<td>2,455</td>
<td>74</td>
<td>2,381</td>
</tr>
<tr>
<td>7:30</td>
<td>2,350</td>
<td>105</td>
<td>2,245</td>
</tr>
<tr>
<td>7:45</td>
<td>1,943</td>
<td>81</td>
<td>1,862</td>
</tr>
<tr>
<td>8:00</td>
<td>2,048</td>
<td>80</td>
<td>1,968</td>
</tr>
<tr>
<td>8:15</td>
<td>1,748</td>
<td>38</td>
<td>1,710</td>
</tr>
<tr>
<td>8:30</td>
<td>1,599</td>
<td>33</td>
<td>1,566</td>
</tr>
<tr>
<td>8:45</td>
<td>1,466</td>
<td>26</td>
<td>1,440</td>
</tr>
<tr>
<td>9:00</td>
<td>1,350</td>
<td>25</td>
<td>1,325</td>
</tr>
<tr>
<td>9:15</td>
<td>1,250</td>
<td>15</td>
<td>1,235</td>
</tr>
<tr>
<td>9:30</td>
<td></td>
<td></td>
<td>24,056</td>
</tr>
</tbody>
</table>
2 automobiles. If 1 lane is used exclusively by buses, the capacity of the remaining 4 lanes is as follows:

5-lane hourly capacity with no buses (8,800 vehicles
- 300 buses + 600 automobiles that replace buses) 9,100
4-lane hourly capacity (7/8 x 9,100) 7,280

Table 5 gives the demand at the toll plaza without buses. This demand and a constant rate of 7,280 other vehicles for capacity were used to determine demand and capacity with respect to time as shown in Figure 7, which is a reduced-scale representation of the original plot. The area within the curve shows the total delay incurred by automobiles and trucks. Because the buses will use the fifth lane and suffer no delay, they are not shown in this plot. From Figure 7, the following information was obtained:

<table>
<thead>
<tr>
<th>Period of delay</th>
<th>Total queuing delay, vehicle-minutes</th>
<th>Maximum delay, min</th>
<th>Maximum number of vehicles in queue</th>
<th>Total queuing delay (194,400 x 1.49), person-minutes</th>
<th>Total running delay (22,447 x 1.49 x 1.1), person-minutes</th>
<th>Total delay, person-minutes</th>
</tr>
</thead>
<tbody>
<tr>
<td>6:30 to 9:35 a.m.</td>
<td>194,400</td>
<td>16</td>
<td>1,950</td>
<td>289,000</td>
<td>36,790</td>
<td>325,790</td>
</tr>
</tbody>
</table>

If an exclusive bus lane is in effect, the increased delay in crossing the bridge will be 223,300 person-minutes, computed as follows:

Delay with bus lane, person-minutes 325,800
Delay without bus lane, person-minutes 102,500
Increased delay, person-minutes 223,300

Figure 7. Relationship between demand and capacity during peak period on east approach of bridge with exclusive bus lane.
Discussion

FRANK S. KOPPELMAN, Tri-State Transportation Commission, New York—The establishment of exclusive bus lanes in heavy travel corridors has received considerable attention by researchers, planners, and administrators seeking ways to improve peak-hour access to central business districts. The study reported by Martin is useful in quantifying some of the effects of providing an exclusive bus lane in one specifically defined circumstance. Unfortunately, the limits of the study as reported by him may lead to erroneous conclusions.

The following comments stress the importance of considering the underlying demand functions that provide the basis for explaining or predicting (or both) changes in observed crossing volumes that will result from changes in the supply function. In general, observed travel volumes represent an equilibrium between travel supply (or cost) functions and travel demand functions. Any change in either of these functions will result in a change in the equilibrium point and, therefore, in the observed travel volume.

The study reported is based on the assumption that a fixed volume of vehicles and persons cross the bridge. In fact, the underlying economic rationale for travel demand indicates that changes in absolute or relative costs—in this case time costs—will lead to changes in observed volume. In the case under consideration, the probable result would be some increase in the bus–automobile modal split (probably also a shift in the time distribution of vehicle arrivals at the bridge, especially automobiles). Rather than make the fixed demand assumption, it is useful to determine the extent of the modal-split shift necessary to reach a break-even point in terms of the established objectives—in this case, reduction in person-minutes of delay—and then to consider the likelihood of such a shift. Such a break-even point occurs at a modal split of 50 to 51 percent on buses. (The author's assertion that person-minutes of delay is always less without the exclusive bus lane, shown in Figure 4, is misleading in that the modal split is itself dependent on the decision of whether or not to provide an exclusive bus lane.)

It is interesting to note that an alternative policy of providing an exclusive bus lane during the period from 7:15 to 8:15 only with priority to buses during the remaining rush period would result in a time delay considerably smaller than that of providing an exclusive bus lane during the entire rush period and would have a break-even result at a modal split of approximately 47 percent (Table 6). The shift to 50 to 51 percent may be unlikely, but a shift in excess of 47 percent is quite reasonable. Although there would be obvious operational difficulties, the benefits might be considerable.

Once the modal-split shift question is introduced, it becomes possible to consider broadening the objectives of the study to consider potential peripheral benefits such as the reduction in CBD congestion. Such peripheral effects may be equally as significant as, or more significant than, the effects of direct time saving.

This discussion indicates some of the reasons for considering the characteristics of vehicle crossing or person crossing demand rather than a fixed-volume condition. An even more comprehensive analysis would consider the shifts in arrival time that might occur as a result of the changes under study. The necessary demand analysis is com-

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### TABLE 6

**COMPARISON OF EXPECTED DELAY TIME**

<table>
<thead>
<tr>
<th>Condition</th>
<th>Delay, person-min</th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Queuing</td>
<td>Running</td>
<td>Total</td>
<td></td>
</tr>
<tr>
<td>Present conditions</td>
<td>66,000</td>
<td>37,000</td>
<td>103,000</td>
<td></td>
</tr>
<tr>
<td>Plan Aa—44 percent on buses</td>
<td>289,000</td>
<td>37,000</td>
<td>326,000</td>
<td></td>
</tr>
<tr>
<td>Plan Aa—51 percent on buses</td>
<td>53,000</td>
<td>37,000</td>
<td>90,000</td>
<td></td>
</tr>
<tr>
<td>Plan Bb—44 percent on buses</td>
<td>134,000</td>
<td>37,000</td>
<td>171,000</td>
<td></td>
</tr>
<tr>
<td>Plan Bb—47 percent on buses</td>
<td>72,000</td>
<td>37,000</td>
<td>109,000</td>
<td></td>
</tr>
</tbody>
</table>

*Based on exclusive bus lane from 6:00 to 9:30 a.m.

b Based on exclusive bus lane from 7:15 to 8:15 a.m. and priority treatment for buses during balance of high-volume period.

c For Plan A, break-even occurs at a modal split of between 50 and 51 percent.

d For Plan B, break-even occurs at a modal split of slightly over 47 percent.
plex but will be ultimately rewarding if it improves the decisions that are made. As part of this analysis, it would be appropriate to consider the development of a probabilistic demand function that could be used to estimate the relative benefits of each alternative over a range of possible conditions. Such a model could be designed to incorporate the generation of nonrecurrent incidents so that their effect on the overall results could be considered.

Although the quantitative approach taken by the author is a significant step forward, an even more rigorous approach is required before discarding a potentially valuable transportation alternative. In any case, the results obtained cannot be readily generalized to other conditions that might lead to quite contrary conclusions.

The author brings up a significant point concerning the relatively limited effect that may be achieved through the establishment of an exclusive bus lane over a short segment of the travel corridor. Future efforts should consider more general possibilities for establishing exclusive or priority lanes over extended portions of the commuter network.
An Empirical Analysis of Lane Changing on Multilane Highways

R. D. WORRALL, Peat, Marwick, Mitchell and Company; and
A.G.R. BULLEN, University of Illinois

The paper describes a macroscopic analysis of lane-changing behavior on multilane highways. It includes descriptions of the pattern and frequency of lane-changing maneuvers observed under varying road and traffic conditions, the distribution of maneuver lengths and times, and the acceptance and rejection of gaps by lane-changing vehicles. Data for the study were collected at a sample of 30 freeway-locations in Chicago.

**LANE-CHANGING BEHAVIOR** may be described in terms of 2 measures: (a) frequency, the number of lane changes occurring among all lanes along a given length of road, \( L \), and over a given time span, \( t \); and (b) pattern, the distribution of lane changes between specific lane-lane pairs along a given road length, \( L \), and over a given time span, \( t \).

More formally, if \( N_{ij}(t_m) \) lane changes occur between lanes \( i \) and \( j \) and within a length \( L \) of an \( n \)-lane roadway during the \( m \)th time interval of length \( t \), then the average frequency of lane changing per unit length per unit time, \( \lambda \), over a series of \( M \) such time intervals may be defined as

\[
\lambda = \sum_{m=1}^{M} \sum_{i=1}^{n} \sum_{j=1}^{n} \left\{ \frac{N_{ij}(t_m)}{L \cdot M \cdot t} \right\} ; \quad N_{ij} \geq 0 \text{ if } i \neq j \quad \text{and} \quad N_{ij} = 0 \text{ if } i = j
\]

In this paper the frequency of lane changing is expressed generally as an average maneuver rate per 500 ft of roadway per minute.

Similarly, the pattern of lane changing may be expressed as an \( n \times n \) transition matrix \( \{P_{ij}\} \), such that

\[
P_{ij} = \left[ \sum_{m=1}^{M} N_{ij}(t_m) \right] / \left[ \sum_{m=1}^{M} V_i(t_m) \right]
\]

where

\[
V_i(t_m) = \text{volume in lane } i \text{ during } m\text{th period } t \text{ and } \quad P_{ij} = 0 \text{ for } i = j.
\]

If the constraint \( P_{ij} = 0 \text{ for } i = j \) is relaxed (i.e., the values on the main diagonal may take on nonzero values) and the value of \( \sum P_{ij} \) is in turn constrained equal to unity, then the matrix \( \{P_{ij}\} \) represents a stochastic matrix, the elements of which represent the probability that a vehicle entering section \( L \) in lane \( i \) leaves that section in lane \( j \) (i.e., the vehicle changes lanes from lane \( i \) to lane \( j \) over length \( L \)).

Data on the frequency of lane changing were collected at 30 locations by means of time-lapse ground photography. The study sample included sections of 2-, 3-, and 4-lane, 1-directional roadways, situated at varying distances from entrance and exit.

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Paper sponsored by Committee on Freeway Operations and presented at the 49th Annual Meeting.
ramp terminals. Mainstream volumes during the study period varied from approximately 1,000 to 5,000 vehicles per hour. Figure 1 shows the location of study sites in the Chicago area.

**FREQUENCY OF LANE CHANGING**

Figure 2 shows a set of typical time-series plots of lane-changing frequency for sections of 2-, 3-, and 4-lane roadway. Equivalent frequency plots for the same locations are shown in Figure 3. In each case, the pattern is essentially a random, apparently stationary variation about a stable mean value. This value—denoted as the average lane-changing frequency $\lambda$ for the section—varies both with location and with flow rate.

For 2-lane roadways, the mean lane-changing frequency varies from 0.7 to 1.4 lane changes per 500 ft per minute. For 3- and 4-lane roadways, the equivalent figures are 1.2 to 3.0 and 2.9 to 3.4 lane changes per 500 ft per minute. These figures are based on data collected at points at least 500 ft from a ramp terminal. The equivalent 1-minute variances follow a similar pattern: 0.6 to 0.9 for 2-lane roadways, 1.8 to 2.9 for 3-lane roadways, and 2.9 to 3.8 for 4-lane roadways.

Figure 4 shows a simple plot of the value of $\lambda$ versus total volume. In this case, the value of $\lambda$ is expressed as the average number of lane changes per 200 ft per minute. The plot suggests that the intensity of lane changing tends to peak at medium-high volume levels and to fall off at both higher...
and lower flow rates. However, to talk of fitting a formal function to the data is clearly inappropriate.

An essentially similar relationship may be discerned between lane-changing intensity and average mainline speed. This is shown in Figure 5. The speed data are average minute speeds, analogous to the volume measurements shown in Figure 4, computed for the total sample of 3-lane locations. The lane-change data are expressed as the average number of lane changes per 500 ft per minute. The relationship is approximately linear in form over the 10-to-40-mph range, and lane changing tends to decrease in intensity as speeds increase above 40 mph.

Figures 6 and 7 show the variation in lane-changing frequency between specific lane-lane pairs as a function of their average minute volume and speed differentials. In each case, $\lambda_{ij}$ is expressed as an average maneuver frequency per 500 ft per minute. The data are again based on the composite sample of 3-lane roadways. Equations based on least squares fits to these data are given in Table 1. As might be expected, lane
TABLE 1
RELATIONSHIPS BETWEEN LANE-CHANGING FREQUENCY AND LANE-LANE VOLUME AND SPEED DIFFERENTIALS ON 3-LANE, 1-DIRECTIONAL ROADWAYS

<table>
<thead>
<tr>
<th>Lane-Lane Movement</th>
<th>Equations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Center to right</td>
<td>( \lambda_{CR} = 0.278 + 0.011 V_{CR} )  ( R^2 = 0.86, \ SE = 0.0048, \bar{x} = 0.308 )</td>
</tr>
<tr>
<td>Right to center</td>
<td>( \lambda_{CR} = 0.074 + 0.012 S_{CR} )  ( R^2 = 0.93, \ SE = 0.0050, \bar{x} = 0.342 )</td>
</tr>
<tr>
<td>Center to left</td>
<td>( \lambda_{CL} = 0.364 - 0.007 V_{CL} )  ( R^2 = 0.24, \ SE = 0.0073, \bar{x} = 0.347 )</td>
</tr>
<tr>
<td>Left to center</td>
<td>( \lambda_{LC} = 0.580 - 0.017 S_{LC} )  ( R^2 = 0.81, \ SE = 0.0041, \bar{x} = 0.315 )</td>
</tr>
</tbody>
</table>

Note: \( \lambda_{CR} \) = average number lane changes per 500 ft per minute between center and right lanes; similarly for \( \lambda_{RC}, \lambda_{LC}, \lambda_{CL}, V_{CR}, V_{CL}, S_{CR} \), and \( S_{CL} \).

Changing tends to be to the lane carrying the lower flow and to the lane with the higher speed.

PATTERNS OF LANE CHANGING

The pattern of lane changing along a portion of multilane road may be represented conveniently by a transition matrix, \( P_{ij} \), the elements of which represent the probability of a single vehicle making a transition, i.e., changing lanes, from lane \( i \) to lane \( j \), within a given distance and a given time span. Figure 8 shows a set of typical transition matrices, calculated for the 6 study locations used in the discussion of lane-changing frequency.

In each case, the individual values in the cells of the matrix represent average transition probabilities computed from a sequence of successive, 15-minute observations of lane-changing behavior at each location. The variance of these successive 15-minute observations is given in parentheses in each cell.

Similar transition matrices were developed for each of the other 26 study locations included in the sample of ground photography. The total set was classified according to study locations and to the average rates of flow during the observation periods and then subjected to a statistical grouping analysis. The results of this analysis are shown for one study location in Figure 9. Similar analyses, with similar results, were performed for each of the other sections of 2-, 3-, and 4-lane roadways studied.
Though the data sample is small (and hence some caution is suggested), results suggest that there is little systematic difference between the lane-changing patterns observed at different flow levels for a given location or geometric configuration.

In the 3-lane example in Figure 9, for instance, medium-volume and very high-volume observations or patterns tend to occur at similar flow levels. The differences that do exist appear to be in terms of the intensity rather than the pattern of maneuvers and to be within rather than between volume groupings. This is particularly noticeable if the values of $P_{ij}$ are readjusted to remove the effects of volume and yield simple estimates of the conditional probability of a lane change occurring between lanes $i$ and $j$, given that a lane change of some sort occurs. Similar results were obtained at each of the other locations studied. In no case did the pattern of lane changing appear to vary systematically with variation in total traffic flow.

**TIME AND DISTANCE REQUIRED TO CHANGE LANES**

The discussion so far has focused on a simple, macroscopic description of lane-changing behavior. Attention is directed here, and in the next section, to a more
detailed examination of the actual mechanics of lane changing, expressed, first, in terms of the time and distance required to complete the lane-changing maneuver and, second, in terms of gap acceptance behavior.

Data Collection

Data for the study were derived from 70-mm aerial photography taken at the points shown in Figure 10. A lane change was considered to commence (Fig. 11) when a vehicle first encroached on the lane line separating the lane in which it was currently traveling from that into which the lane-change maneuver was to be made. The maneuver was considered to be completed once the vehicle had completely crossed that line. The remaining head and tail portions of the maneuver were then analyzed separately. The head portion of the maneuver is the time and distance required for a vehicle to move from a straight-ahead path in its origin lane to first intercept the dividing lane line, and the tail portion of the maneuver is the equivalent time and distance required for a vehicle to return to a straight-ahead path after crossing that line.

Numerical Results

Figure 12 shows the relationships between volume, speed, and maneuver time; Figure 13 shows the relationships between volume, speed, and maneuver distance.
The functions $T = h(q, v)$ and $L = g(q, v)$ (where $T$ = maneuver time, $L$ = maneuver length, $q$ = volume, and $v$ = speed) are represented on the figures by families of parabolic curves, each solid "contour line" representing the locus of points of either constant maneuver time or maneuver distance. The range of the data set, i.e., the range of traffic conditions studied, precluded development of a complete contour map for either maneuver length or maneuver time for all traffic conditions. The existence of a parabolic relationship between maneuver length and time and between traffic volume and speed is, however, strongly suggested by the data shown in Figures 12 and 13.

Figure 14 shows the results of an analysis of the head and tail portions of the lane-changing maneuver. The average time required for the head of the maneuver was 1.25 seconds with a standard deviation of 0.4 second and for the tail, 1.95 seconds with a standard deviation of 0.5 second. Figure 15 shows the equivalent distances. The mean distance for the head was 110 ft and for the tail, 160 ft.

The distribution of lateral placement for the changing vehicle prior to and after the lane-changing maneuver is shown in Figure 16. Vehicle placements are relatively widely distributed with respect to the centerline both before and after the maneuver. The average vehicle moved laterally 27 ft before encountering the lane line and 30 ft after crossing the line before regaining a straight travel path. The average lateral speed of the changing vehicle during the head of the maneuver was thus 2.2 ft per sec, and during the tail, 1.5 ft per sec. For the total maneuver, including the head and the tail, the average lateral speed of the changing vehicle was 3.1 ft per sec, reflecting the higher average lateral speed achieved during the core of the maneuver as the vehicle was actually crossing the lane line.

**AN ANALYSIS OF MINIMUM ACCEPTABLE GAPS FOR LANE CHANGING**

The following 3 definitions are used in this analysis:

1. Accepted Gap—the time (or distance) headway between the leading and lagging vehicles that defines the gap accepted by the maneuvering vehicle and is measured at the start of the maneuver;
2. Accepted Lead—the time (or distance) headway between the maneuvering vehicle and the leading vehicle in the destination lane measured at the start of the maneuver; and
3. Accepted Lag—the time (or distance) headway between the maneuvering vehicle and the lagging vehicle in the destination lane measured at the start of the maneuver.
Table 2 gives the values of the 95 percentile minimum gap, lead, and lag values observed in the field. The data are based on a total of 1,706 lane changes observed under varying traffic conditions and are stratified by lane density for the destination lane and relative maneuver speed measured between the maneuvering vehicle and the leading-lagging vehicles defining the destination gap. The 95 percentile value was selected somewhat arbitrarily as representative of the average minimum value for a given traffic condition. The observed values of minimum accepted time gap, lead, and lag vary respectively from 1.1 to 2.9 sec, 0.2 to 0.7 sec, and 0.3 to 0.6 sec. The equivalent distance gap, lead, and lag headway measures are respectively 86 to 341 ft, 13 to 80 ft, and 14 to 76 ft.

No stable relationship is discernible between the observed minimum accepted gap, lead, or lag values and the traffic density in the destination lane. Minimum accepted gap, lead, and lag values in both time and space, however, tend to increase as the relative maneuver speed increases. This increase is most marked in lead rather than lag values. As might be expected, the minimum accepted gap size is greater for a given traffic condition than the sum of the equivalent minimum accepted lead and lag values, suggesting that either the lead or the lag value may be critical in any given maneuver but probably not both.

**FORMULATION OF AN ACCEPTANCE FUNCTION FOR LANE CHANGING**

Unlike most other gap acceptance and rejection situations, it is not possible in the case of a lane change to identify a rejected gap directly. Some mild computational gymnastics must be resorted to, therefore, in order to generate an empirical acceptance function.

Assume that the size of the gap immediately available to a driver in an adjacent lane in no way influences his initial desire to change lanes, and that the probability of his wishing to change lanes remains constant for a given set of environmental conditions. These conditions might include roadway geometrics, overall volume of traffic, or the relative speeds between given lanes of traffic. Assume also that gap acceptance behavior as observed in the field is an unbiased sample of the behavior of all drivers under a given set of environmental conditions, and that an unbiased estimate may be made of the frequency distribution of available gaps in each adjacent lane.
Then one may write

\[ r(t) = a(t) \cdot p \cdot F(t) \]

where

- \( r(t) \) = the percentage of gaps accepted that are of size t,
- \( a(t) \) = the percentage of size t gaps available to all drivers,
- \( p \) = the percentage of available gaps of all sizes into which a lane change is desired, and
- \( F(t) \) = the percentage of drivers who will accept a size t gap.

The formulation can be transposed and \( F(t) \) can be expressed as

\[ F(t) = \frac{r(t)}{a(t)} \cdot p \]

The parameters \( r(t) \) and \( a(t) \) can be directly measured through sampling techniques applied to actual freeway operations. The parameter \( p \), however, remains unknown. However, because we assume that \( p \) is constant, we may consider 2 gap sizes, \( t_1 \) and \( t_2 \), and compute

\[ F(t_1) - \frac{r(t_1)}{a(t_1)} \cdot p \]  
\[ F(t_2) = \frac{r(t_2)}{a(t_2)} \cdot p \]

The ratio of \( F(t_1) \) to \( F(t_2) \) can be written

\[ \frac{F(t_1)}{F(t_2)} = \frac{\frac{r(t_1) \cdot a(t_2)}{a(t_1)}}{\frac{r(t_2)}{a(t_2)}} \]

Let

\[ N(t) = \frac{r(t)}{a(t)} \]

Then

\[ \frac{F(t_1)}{F(t_2)} = \frac{N(t_1)}{N(t_2)} \]

If there exists some gap size \( t_0 \), which we can assume to be acceptable to all drivers, then at this value \( t = t_0 \) the acceptance function \( F(t_0) \) will take on the value of 1.0, i.e., \( F(t_0) \) represents the horizontal asymptote for the acceptance function. Therefore

\[ \frac{F(t)}{F(t_0)} = F(t) = \frac{N(t)}{N(t_0)} \]

Determination of the acceptance function \( F(t) \) thus requires estimation of the values of \( r(t) \) and \( a(t) \). Given these values, one may determine, at least conceptually, a functional form for the gap acceptance curves and, hence, compute its mean and variance. Comparison of these values for functions developed for different highway and traffic conditions then provides a convenient basis for evaluating gap acceptance behavior within the lane-changing process.

The time and distance measurements of headway, lag, lead, and gap sizes used in this analysis are shown in Figure 11. All distance measurements are made at the outset of the maneuver. The time measures are then calculated as follows:
Time headway for changing vehicle in origin lane = distance headway/speed of changing vehicle
Time lag for changing vehicle in destination lane = distance lag/speed of lagging vehicle
Time lead for changing vehicle in destination lane = distance lead/speed of changing vehicle
Time gap for changing vehicle in destination lane = distance gap/speed of lagging vehicle

Comparison of the speeds of the leading and lagging vehicles in the destination lane indicated that the values did not differ significantly for any of the traffic conditions studied.

The data set was first stratified according to the traffic density in the destination lane and the direction of the maneuver; i.e., lane changes to the left or to the right were separated. The availability of gaps in the adjacent lane was initially estimated on the assumption that the positioning of vehicles in adjacent lanes was independent (i.e., that it was equally likely for a vehicle in the lane adjacent to the maneuvering vehicle to be positioned at any point within the available gap). The availability function was calculated as

\[
P(\text{gap} \leq T_K) = \frac{\sum_{i=1}^{K} t_i n_i}{\sum_{\text{all } j} t_j n_j}
\]

and

\[
P(\text{lag or lead} \leq T_K) = \frac{\sum_{i=1}^{K} t_i n_i + t_K (\text{number of gaps} > t_K)}{\sum_{\text{all } j} t_j n_j}
\]

where

\[
t_i = \text{gap of size } t_i \text{ and } n_i = \text{number of times } t_i \text{ is observed.}
\]

The assumption of independence between lanes, however, is only weakly supported by empirical analysis. In view of this, a second method was developed to estimate the availability distribution, \( a(t) \), based on the simplistic stratification of the data set according to the distance headways and relative speeds of the vehicles involved in the lane-changing maneuver.

The asymptote to the presumed acceptance function \( t_\alpha \) was estimated by first discounting all but the lower 95 percent of the 2 distributions \( r(t) \) and \( a(t) \). The values of \( r(t) \) and \( a(t) \) were then aggregated successively for each interval in the distribution from \( n \) to \( n - K \) until the following inequality was satisfied:

\[
\frac{\sum_{i=1}^{n} r(t_i)}{\sum_{i=1}^{n-K} a(t_i)} \geq \frac{r(t_j)}{a(t_j)} \quad \text{for all } j \leq (n - K)
\]
A set of gap acceptance functions based on the entire data set considered as a whole is shown in Figure 17. Figure 18 shows an equivalent set of acceptance functions based on a 5-level density stratification. The results of a probit analysis applied to the entire data set are shown in Figure 19. The estimated equation is

\[ Y = 1.523 + 1.513 \times X \]

where

\[ X = \log_e [\text{gap size}] \] and
\[ Y = \text{probit of } F(t). \]

The mean accepted gap size derived from this analysis is 1.985 seconds. The equivalent lead and lag values are 0.527 second and 0.676 second.
THE INFLUENCE OF RAMP TERMINALS AND HIGHWAY DESIGN ON LANE CHANGING

In an attempt to assess the influence of ramp terminals on lane-changing behavior we made estimates of the average lane-changing rate, \( \lambda \), at varying distances from a sample of exit and entrance ramp terminals. Separate data were collected for 2-, 3-, and 4-lane, 1-directional roadways and for varying volume levels. For each location, the data were tested for any systematic bias or nonrandom characteristic or both that could be related to the design of the highway or its environs. Similar analyses were also performed systematically for lane-changing patterns by the use of the transition matrix notation described earlier. Figures 20, 21, and 22 show the results of these analyses respectively for 2-lane, 3-lane, 4-lane roadways.

The sparseness of the data for a given location and geometric configuration and the absence of any significant experimental control over potentially confounding influences makes it difficult, if not impossible, to draw very meaningful conclusions from these figures. In each case there is a tendency, albeit weak and nonsystematic, for the intensity of lane changing to increase in the vicinity of the ramp terminal. This is most marked on 3-lane roadway sections and much less marked on 2- and 4-lane sections. The variation about this trend, however, is extremely high. In no case are there grounds for developing a formal relationship between average lane-changing frequency and distance to and from ramp terminals. Variations in mainstream volume do not appear to exert any systematic effect on the spatial distribution of maneuver intensity.

Examination of the distribution of minute-by-minute maneuver counts at each location confirms the general results reported earlier. The distribution of maneuvers was again essentially random at all except 3 locations. These 3 were each immediately downstream from an entrance ramp. Examination of the distributions for these locations indicated that the major discrepancy between the observed and the theoretical random distributions could in each case be attributed to the effect of intermittent platoons of vehicles entering the freeway via the upstream on-ramp.

The general, if not very conclusive, finding is confirmed by the variation in \( \lambda \) along a section of approximately 1,500 ft of the Edens Expressway in Chicago (Fig. 23). The figure shows the profile of a gradual increase in the lane-change rate on the approach to the interchange, a decrease within the interchange area, and a sudden increase immediately downstream of the entrance ramp.

Examination of the equivalent patterns of lane changing, expressed as transition matrices \( P_{ij} \), yielded equally inconclusive results. Again, the pattern of maneuver tended to reflect the proximity of the ramp.
terminal—indicated in this case by an increase in the values of $P_{ij}$ associated with movements into and out of the lane adjacent to the ramp terminal. Again, however, this trend was not sufficiently marked to enable formal distinction to be drawn among the patterns observed at varying distances from entrance and exit terminals.

In interpreting these results, one should recognize that lane-changing maneuvers are generated for a wide variety of reasons and that their patterns and intensity are influenced by many different factors. Ramp location is only one such factor, albeit an important one. Of at least equal importance are the quality and location of directional signing, the location of adjacent ramp terminals, the volume of truck traffic, the relative speeds in the adjacent traffic lanes, and the aggressiveness of individual drivers. All these and more contribute significantly to the pattern and intensity of maneuvers observed at any one point in either time or space. Given the systemic, and rather crude, viewpoint adopted here and the absence of any significant degree of experimental control, it is not surprising that the results obtained were relatively bland and inconclusive.

Equally important, however, is the fact that no generally dominant influence of ramp terminals on lane-changing behavior was detected at the level of analysis pursued here. Given that most, if not all, of the locations studied represented good rather than bad examples of geometric design, this suggests that lane-changing behavior is not seriously influenced by the proximity of ramp terminals, and particularly that the intensity of maneuvers is not increased by ramps spaced at the intervals (roughly $\frac{1}{2}$ to $1\frac{1}{2}$ miles) observed in the field.

**SUMMARY AND CONCLUSIONS**

This paper has presented a selection of elementary empirical results concerning lane-changing behavior on 1-directional, multilane roadways. The discussion has ranged from considerations of the frequency and pattern of lane changing to an analysis of the mechanics of the actual lane-changing maneuver and the effects of ramp location on lane-changing behavior. All of the analyses have been based on field data collected in the Chicago area.

Lane changing is shown to be essentially a random event in the traffic stream, subject to considerable variation at any point in either time or space. Average frequencies and patterns of maneuver exhibit a weak, but systematic, relationship to traffic and roadway conditions. Maneuver times and lengths similarly tend to vary as a function of the volume and speed of traffic. Gap acceptance behavior displays only a very weak, and as yet largely undefined, relationship to traffic flow. Although the average rate of lane changing varied considerably across the set of traffic conditions and field locations included in the study, the distribution of maneuvers over time remained random in virtually every case. The major exceptions to this finding were study locations situated immediately downstream from high-volume entrance ramps. There was no indication that the overall pattern or intensity of lane changing changed from random to nonrandom at a given distance from an entrance or exit terminal.
A final word of caution is in order. Although these findings are based on the analysis of a relatively large and well-defined data base, the amount of substantive information underlying some of the conclusions and the range of traffic and design conditions for which these conclusions are valid without extrapolation are relatively small. A truly definitive analysis here should be based on a much larger sample of study locations. In all cases, it should be borne in mind that only a very crude experimental control could be exercised over the collection and analysis of the data and that all of the results pertain to data collected in the Chicago area; no information was obtained, for example, on lane changing on grades where the results may be very different from those obtained here for level terrain. Equally, no attempt was made to explain why lane-change maneuvers occurred but only to describe some of their characteristics. Clearly, this analytical perspective is reflected in and may partially bias the results discussed here.

ACKNOWLEDGMENT

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REFERENCES

An Evaluation of Ramp Control on the Harbor Freeway in Los Angeles

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This paper describes a ramp control project on the 8-lane Harbor Freeway in Los Angeles. Congestion occurred each weekday in the 4 outbound lanes during the evening peak period, while the adjacent parallel streets were relatively uncrowded. The control project is designed to reduce total travel time through the freeway corridor by metering 5 on-ramps and closing 1 ramp in a 5-mile section during the peak period.

Some unusual techniques are being employed: Buses are allowed to bypass ramp queues and to make left turns where other traffic cannot; storage of queued vehicles on a frontage road is carried back across an intervening major thoroughfare by timing the intersection signal so that the intersection itself remains clear; and, at one location, storage room is increased and high metering rates are made possible by the release of vehicles 2-abreast at the ramp signal.

Results show that freeway users are saving about 1,000 vehicle-hours per day against a loss, or increased travel time for diverted or delayed ramp traffic, of about 130 vehicle-hours.

IN SEPTEMBER 1968, the California Division of Highways, with the cooperation of the Los Angeles City Traffic and Police Departments, began a freeway ramp control project on the southbound Harbor Freeway in south Los Angeles. The Harbor Freeway is a major 8-lane facility extending 22 miles from downtown Los Angeles to San Pedro. Basically, the project involves limiting on-ramp traffic in a 5-mile section of the 4 southbound lanes during evening peak hours (Fig. 1). The purpose is to reduce delay to freeway users and gain optimum use of the freeway and street system during the rush hours.

The science of controlling ramp traffic to reduce total travel time (freeway and surface streets) is comparatively new. However, the Harbor Freeway project is not the first project of this type. A similar, although smaller, system has been in effect in Los Angeles on the Hollywood Freeway for more than a year; other projects are in operation in San Diego, Chicago, Detroit, and Houston. The Harbor Freeway was selected as the first major effort in Los Angeles because of the high probability of success. The availability of relatively uncrowded parallel streets in the Harbor Freeway corridor and excess capacity on the downstream freeway sections were promising indicators that more good could be done more quickly here than anywhere else in the Los Angeles area. Also, several on-ramps in this area adapted themselves rather easily to metering (i.e., available frontage road and storage area). Control of 6 on-ramps to the southbound Harbor Freeway is accomplished by various means that were determined by an in-depth study requiring application of basic traffic engineering principles at each ramp. One
ramp is closed during the control period by the use of barricades, 2 ramps are controlled by platoon metering, and 3 ramps are controlled by single-car metering. Generally, the control period is from 3:45 to 5:45 p.m. each weekday.

This paper is primarily an evaluation of the ramp control project, but it also briefly describes the planning prior to implementation. The conclusions drawn are based on extensive data collected before and after the project was initiated.

CONDITIONS BEFORE CONTROL

Freeway

Prior to the ramp control, congestion was caused by simply too much traffic trying to use the southbound Harbor Freeway south of Exposition Boulevard. Typical weekday operation of the freeway before control is shown in Figures 2, 4, and 5, representing density, speed, and volume. (Figure 3 shows density after control.) Congestion normally started about 3:45 and lasted until after 6:00 and extended from Manchester Avenue to just north of Adams Boulevard.

The density contours (Fig. 2) indicate 2 distinct congested areas—1 south and 1 north of Slauson Avenue. The congestion was the result of excess demand of approximately 500 vehicles per hour (vph) affecting 2 key sections. The most southerly bottleneck was where the Florence Avenue on-ramp traffic joined an almost full freeway and caused excess demand for the section. This caused congestion that extended to and upstream of Slauson Avenue. The second major bottleneck was caused by Vernon Avenue on-ramp traffic joining a near-capacity flow to create excess demand for an upgrade (800 ft at approximately 5 percent) approaching Slauson Avenue.

Total delay on a typical weekday was about 60,000 vehicle-minutes from 3:45 to 6:15 p.m. Maximum delay to individual vehicles was more than 6 min upstream of Slauson Avenue and about 3 min between Slauson and Manchester Avenues.

Figure 4 shows travel times and speeds throughout the evening period; travel times before control were frequently greater than those shown. Average speeds for the entire trip from 30th Street to Century Boulevard were as low as 15 mph, with much stop-and-go driving. A freeway trip from 30th Street to Century Boulevard frequently took about 17 min during the peak period.

Typical peak-period flow rates, shown in Figure 5, represent conditions from about 5:00 to 5:30 p.m. At this time, the entire length upstream is controlled by the capacity at the most southerly bottleneck. For this reason, throughput upstream of Manchester Avenue was considerably below capacity of the various sections. Before 5:00 p.m.,
Figure 3. Density after controls on the southbound Harbor Freeway from 3:30 to 6:30 p.m., Wednesday, October 16, 1968.
conditions were somewhat similar, except that the Slauson Avenue grade was an independent bottleneck with flow rates of about 7,100 vph.

**Surface Streets**

Preliminary data indicated that the parallel streets, Broadway and Figueroa, would be the best alternate routes for diverted traffic. Both of these are major thoroughfares. Broadway has 4 lanes, a painted median, and left-turn pockets. Figueroa has the same and, in addition, a parking lane on each side with restricted parking during peak hours.

![Figure 4. Travel time and speed on the southbound Harbor Freeway between 30th Street and Century Boulevard (5.08 miles).](image)

![Figure 5. Typical peak-period flow rates from approximately 5:00 to 5:30 p.m.](image)
westbound 37th Street because these turning movements would block eastbound 37th Street traffic. The metering rate of 650 to 850 vph is too great for single-car metering. Platoon metering was the answer, and it has worked well. The 37th Street on-ramp was particularly adaptable to platoon metering because of a long collector road. This collector road also handles off-ramp traffic that tends to break up platoons and approximate single-car metering.

Santa Barbara Avenue

The Santa Barbara Avenue on-ramp (Fig. 7) had to be closed to all traffic, except buses, because of inadequate storage for metering.

Vernon Avenue

The Vernon Avenue on-ramp (Fig. 8) has a minimum storage area, so normal metering at the ramp would cause interference with Vernon Avenue traffic. Closure of the ramp would be too restrictive. Therefore, it was necessary to devise a method for control that allowed approximately the existing volume to enter without an extensive queue forming at the ramp. Metering signals were placed on the west frontage road just upstream of the Vernon Avenue off-ramp. These signals intercept traffic coming from Santa Barbara Avenue and release an amount slightly below the metering rate at the Vernon Avenue ramp signal. This allows the right turn from eastbound Vernon Avenue to reach the on-ramp. Initially, the left turn from westbound Vernon Avenue was prohibited because the ramp queue might have kept this traffic from clearing the intersection. Now the left turn has a separate green phase and pocket lane. Traffic on the west frontage road approaches in 2 lanes and is released in 2-lane platoons from the front-
Average speeds on both Broadway and Figueroa were 15 to 30 mph. A trip from 30th Street to Century Boulevard via Figueroa took an average of about 15 min and a peak of 19 min. Hourly volumes on southbound Broadway ranged from 1,200 vehicles at 57th Street to 850 vehicles at 85th Street. On Figueroa, volumes were as high as 1,400 vehicles at 57th Street and diminished to about 1,100 vehicles at 85th Street.

PLANNING

The basic theory used in planning was a demand-capacity analysis on the freeway. For 2 bottlenecks that were located, capacity was determined, demand was estimated, and a ramp control plan was developed that would reduce demand to what each section could handle. A more comprehensive discussion of the theory of ramp control is presented in another paper (1).

An origin and destination survey of traffic using the ramps within the critical area was necessary for an accurate prediction of the pattern of traffic diversion caused by control. We were able to predict that a high percentage of the traffic subject to control would enter at upstream locations and that parallel facilities would not become overloaded.

During the planning phase, meetings were held with interested individuals and local agencies to inform them of our efforts. The ramp users were informed by handouts the week before control began. Alternate route signs were placed on the city streets to aid the diverted traffic.

One of the most important phases of the project was planning the method for control at each ramp location to determine the amount of traffic that should be let on and the means for minimizing the effect of control on "innocent" traffic. When severe metering was necessary, we knew that time and not distance would determine the queue length. In other words, the driver is concerned with the amount of time it takes him to enter the freeway, rather than with the distance he must travel. Therefore, if a short queue were desired, then a more restrictive metering rate had to be used. Increasing the delay to the individual motorist causes him to seek an alternate route.

The metering signals operate on a fixed-time basis with up to 3 different rates operated by a time clock; a 3-dial controller is used. The rates are set based on "typical" conditions or traffic patterns. The system is not traffic-responsive. In the following is a discussion of some of the major points considered in planning the controls at each ramp.

37th Street

The 37th Street on-ramp (Fig. 6) was the most critical on the project. It is the first on-ramp south of the Santa Monica Freeway and as such has a very high demand (1,200 to 1,300 vph). Limiting this traffic to 650 to 850 vph could have very seriously affected nonfreeway traffic on 37th Street and Flower Street, and possibly Exposition Boulevard and Figueroa Street. This problem was minimized by the prohibition of left turns into the ramp from

Figure 6. 37th Street on-ramp where platoon metering is used because of high metering rates used. Most traffic comes from southbound Flower Street and is stored in the left-turn lane upstream of Exposition Boulevard. Left turns to the ramp from westbound 37th Street are prohibited because they would interfere with eastbound 37th Street traffic. Right lane of the ramp is striped and signed for buses and emergency parking only. Congestion in northbound lanes is the result of a previous incident in the northbound lanes and is not affecting southbound traffic at this time.
age road metering signals. These metering signals are interconnected with the signals at the Vernon Avenue and west frontage road intersection. The green phases of the two sets of signals coincide so that metered traffic does not have to stop at Vernon Avenue.

In order to handle these two lanes of traffic, the Vernon Avenue on-ramp entrance was widened to 2 lanes. The ramp metering signals release 1 car at a time from each lane. The ramp signals are located near the entrance from the frontage road leaving most of the ramp available for the 2 cars to merge into 1 lane before they reach the freeway acceleration lane. Initially, it was difficult to maintain single-car operation, but this was partially resolved through the use of signs that read METERED—ONE CAR PER GREEN THIS LANE. With these signs, violations are around 10 percent; without the signs, they were 15 to 20 percent.

Slauson, Gage, and Florence Avenues

On the basis of traffic volumes before control at Slauson, Gage, and Florence Avenues, metering appeared to be necessary at these ramps only after 5:00 p.m. However, diversion from upstream ramps before 5:00 p.m. was anticipated (which in fact happened), so the metering periods at these ramps were set to approximately coincide with those for upstream ramps (3:45 to 5:45 p.m.). This holds the ramp volumes to about the same as they were before control.

Manchester Avenue and Century Boulevard

On-ramp volumes at Manchester Avenue and Century Boulevard were not so high that they had to be metered; however, signals were placed at each in anticipation of an increase from diverted traffic. Volumes have increased, but not to the point that metering is necessary.

Buses

Alterations were made to provide special access for buses. At 37th Street (Fig. 6), the right lane of the ramp was striped and signed for buses only. This allows them to bypass the ramp queue. The ramp configuration at Santa Barbara Avenue, where buses have always had a separate entrance to the ramp (Fig. 7), made the access of buses and prohibition of other vehicles quite easy.

CONDITIONS AFTER CONTROL

Freeway

The before-and-after density data (Figs. 2 and 3) were used to estimate freeway travel times. These data give a good picture of the amount of congestion between Adams Boulevard and Manchester Avenue on typical days. There is considerable congestion upstream of Adams Boulevard, and this has not changed much with control.

Total freeway travel time before control from 3:45 to 6:15 p.m. between Adams Boulevard and Manchester Avenue was 188,000 vehicle-minutes per day. Now during the same period, freeway travel time is approximately 128,000 vehicle-minutes, a reduction of 60,000 vehicle-minutes in spite of an increase in the total vehicle-miles of travel.

During the period before control, there were 81,000 vehicle-miles of travel on the freeway. This means that during the total 2½-hour period average speed between Adams Boulevard and Manchester Avenue was 2.32 min per mile (188,000 divided by 81,000) or 25.8 mph. During the period after control, there were approximately 86,000 vehicle-miles of travel, and average speed was 1.49 min per mile or 40.3 mph. Without the extra vehicle-miles of travel, the after travel time would have been 81,000 times 1.49 or 121,000 vehicle-minutes. Savings on the freeway, therefore, were actually 67,000 vehicle-minutes for the base or original amount of travel instead of the calculated difference of 60,000 vehicle-minutes.

The extra travel on the freeway is caused by about 900 vehicles that now enter the freeway upstream of the control section instead of at the controlled ramps as they for-
merly did. These trips should add to the savings, although to determine their actual savings is virtually impossible. Probably, where there is enough backtracking, some of these trips actually lose time compared to the time they took before control. However, we are sure the net result is added savings because (a) the great majority of these extra trips originated north of the control section and (b) their responses to a survey questionnaire indicate they save time. Net savings are not as great as noted, however, because there is delay to diverted traffic and to vehicles at the ramp metering signals.

Approximately 500 vehicles no longer use controlled ramps where delay may be experienced. They are staying on surface streets to Manchester Avenue or Century Boulevard, or staying off the freeway completely. These trips will average about 2 minutes longer than they formerly did on the freeway for a total delay of 1,000 vehicle-minutes. This is a conservative estimate as it assumes that all this traffic starts upstream of 37th Street and goes all the way to Century Boulevard, but, actually, some of it is on streets a lesser distance and would not suffer as much delay.

Queue lengths versus time were plotted for each controlled ramp on a typical day. With these, the delay caused by the signals was estimated for each on-ramp. Total delay at the ramps is about 9,000 vehicle-minutes per day. The net savings then to traffic in the freeway corridor is at least 50,000 vehicle-minutes per day from the ramp control.

Generally speaking, individual motorists who enter the freeway at 37th Street or Vernon Avenue save time even though they must wait at the metering signals. They more than make up their lost time at the signal by increased speeds on the freeway. Those who enter at Slauson or Gage Avenue break even; their wait at the ramp is about the same as their gain on the freeway. Those entering at Florence Avenue are the only ones who actually lose time because they were entering at the head of the line before control.

Figure 4 shows travel time and speed for individual trips through the control section. These were obtained by photographic methods (taking pictures of traffic at points along the freeway and matching cars in the pictures to get travel time) and floating car runs. The before data were collected during the summer of 1968 and represent somewhat less congestion than that indicated by the before densities, which are for a day in October 1966. After control, average speed is frequently more than 40 mph. Before control, a trip from 30th Street to Century Boulevard off-ramp could have taken as long as 16 to 17 min (even though the densities do not show this). This same trip now takes a maximum of 7 to 9 min. Conditions upstream of 30th Street have remained about the same.

Before control, about 6,300 vehicles entered the freeway from the 6 controlled ramps between 3:45 and 5:45 p.m. With the Santa Barbara Avenue ramp closed and the others metered, now about 4,900 vehicles are allowed to enter.

Increased input to the control section from the freeway upstream indicates about 900 of the diverted vehicles now enter the freeway upstream of the control. These motorists are probably not backtracking to enter the freeway, that is, their trips originate upstream of the control area. An origin-destination survey made prior to control showed that at least 32 percent of the Vernon Avenue on-ramp traffic would probably have entered upstream had it not been for the freeway congestion. These motorists obviously feel they are saving time if they have changed their routes. It is estimated that now 500 vehicles either stay off the freeway altogether or use a ramp downstream of the controls.

As expected, off-ramp volumes have decreased somewhat since control. Before control, 45 percent of the traffic exiting at Vernon Avenue alone continued south along the freeway corridor. Other off-ramps were somewhat similar. These motorists were either re-entering the freeway downstream or staying off completely to avoid freeway congestion until their desired destination.

Figure 5 shows typical peak-period flow rates on the freeway. These volumes are representative of the period from 5:00 to 5:30 p.m. when the greatest throughput increase is realized. During this period, the downstream bottleneck at the Florence Avenue on-ramp controlled the before period throughput of all upstream sections including the Slauson Avenue grade.
Capacity of a bottleneck cannot change, except in very special cases and then only to a very limited degree. Although there is no conclusive evidence, capacity of the Slauson Avenue grade may have increased slightly because of increased approach speeds, but the rate increase from 6,600 to 7,150 vph shown in Figure 5 does not represent this increase. The low before rate was a result of the downstream bottleneck controlling output during this period.

No doubt, the best overall picture of the changed traffic conditions is the before-and-after densities shown in Figures 2 and 3. The after densities indicate that, before 5:00 p.m., there are only isolated points of densities over 50 vehicles per mile per lane (vml) with the major portion of operation below capacity. This slack allows a natural recovery capacity to dissipate congestion resulting from incidents. After 5:00 p.m., there are still pockets of congestion upstream of both bottleneck sections. Initial metering rates did not eliminate this congestion because they were based on a precontrol condition where input to the section dropped significantly after 5:00 p.m., and off-ramp volumes increased. This condition changed once control began. Input now stays at a high level until 5:20 p.m., and off-ramp volumes actually decreased during this part of the control period.

Surface Streets

Subjectively, traffic conditions on the city streets have not changed appreciably. Trips through the control area on either Broadway or Figueroa Street average about the same time as they did before. Speeds are between 15 and 30 mph. Speeds on the surface streets were obtained by a license plate study that involved recording time and license plate number at key locations. These were then matched to determine elapsed time. Floating car runs were used to supplement these data.

Not all the diverted traffic could be located with the techniques available. No doubt some motorists are using routes other than Broadway or Figueroa Street. However, based on origin-destination data, street patterns, and travel times, Broadway and Figueroa Streets are the best alternate routes for most of the diverted traffic. The effect on these routes was barely measurable with travel times and volume counts.

An estimated 500 vehicles have been diverted from the freeway throughout the control period. Traffic volumes, obtained at various locations on Figueroa Street and Broadway, show that the volume has not significantly increased at any one location. Five hundred vehicles spread over a 2-hour period and throughout the adjacent streets have easily been absorbed.

RESULTS AND CONCLUSIONS

The project is very successful. Delay to the freeway motorist is significantly reduced, delays to controlled traffic are relatively minor, and travel time on adjacent streets has shown little or no increase. Basically, the ramp control prevents about 1,400 vehicles, which used the freeway during the before period, from entering the on-ramps within the control area from 3:45 to 5:45 p.m. Of these, approximately 900 vehicles now enter the freeway at some ramp upstream, and 500 vehicles are being diverted to surface streets.

Congestion on the southbound Harbor Freeway is almost entirely eliminated south of Exposition Boulevard, except on days when incidents occur and reduce capacity. A minimum average speed of 40 mph is now maintained throughout the control period—with occasional shock waves. This compares with 15 to 25 mph under stop-and-go driving conditions before control. Individual motorists save a maximum of 9 min per through trip on some days, and the average motorist saves 4 to 5 min per trip. The controls benefit not only through traffic, but also drivers that get off the freeway in the control area, depending on where they exit.

Traffic conditions on the parallel city streets, principally Broadway and Figueroa, are practically unaffected by the ramp control. Total travel times and volumes through the control area on either Broadway or Figueroa Street have remained nearly the same.

The net effect of the control project is estimated to be a time reduction on the freeway of about 60,000 vehicle-minutes and increased travel time of roughly 10,000 vehicle-
minutes per day to diverted traffic and traffic waiting at the metering signals. This assumes that travel time has not significantly increased for motorists who have always used city streets. Drivers who formerly entered the freeway in the control area and now enter upstream also save time, but the amount is unknown.

Solicited opinions of users at the Manchester Avenue and Century Boulevard off-ramps show the motorists to be very much aware of, and in favor of, the system. Ninety-two percent of the responses indicated approval. Those entering at the controlled ramps were not as enthusiastic as those entering upstream, but the majority still approved the project. Figure 9 shows a typical response.

The project shows that a relatively small number of vehicles in excess of capacity at key bottlenecks can cause severe and recurrent congestion. In this case, the excess was about 500 vehicles over a 2-hour period.

Significant knowledge has been gained about ramp control from this project. The following are the important points:

1. Planning of control methods is very important and, in this project, was directly responsible for minimum disruption to surface street traffic. The slight sacrifices in the optimum freeway control that sometimes must be made to ensure good street operation are not critical.

2. Platoon merging or random merging of single vehicles has caused no problems.

3. Incidents are very frequent. The entire 5-mile section at times is operating virtually at capacity and any incident in this reach has a drastic effect. When operating at capacity, the storage built up by each incident cannot be dissipated for the rest of the control period. The sharp reductions in metering rates that would be required to dissipate congestion are usually not possible because of the severe congestion that would be caused on surface streets. In fact, unless good information can be given to drivers approaching metered ramps, we do not feel a sharp and unpredictable (to the driver) fluctuation of metering rates is advisable. Because of the frequency of incidents and difficulty in dissipating resultant congestion, operating at volumes slightly below capacity, if possible, is probably justifiable. This slack allows a natural recovery capacity. In

Figure 9. Survey response of driver that exited at Century Boulevard about 5:20 p.m.
our project this is possible from 3:45 (start of control) to 5:00 p.m. We now operate with some slack during this period. The adjacent streets are at or near capacity only from 5:00 to about 5:30 p.m. After 5:00 p.m. the metering would have to be too restrictive to operate with any slack.

4. This is not a traffic-responsive system. Such a system would provide some additional benefits. However, for reasons we have mentioned, more refined metering rates is not one of them. The primary benefit would be to take advantage of unused capacity downstream of incidents and to allow for major changes in input demands. In other words, we could start and end control at different times more in keeping with actual freeway conditions.

REFERENCE

The Analysis and Design of Freeway Entrance Ramp Control Systems

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This paper describes some of the results obtained from an analytic study undertaken as part of the Gulf Freeway Surveillance and Control Project in which the on-line, dynamic control of individual entrance ramps is investigated. For this study 2 alternate control philosophies are considered. The first philosophy concerns the control of ramps on which a string of one or more vehicles is released when a suitable merge opportunity arises and when all previously released vehicles have successfully merged into the freeway stream. The second control philosophy concerns ramps on which the requirement that the ramp be cleared of all previously released vehicles is relaxed to increase the merge capacity of that ramp. In this second case, the controller may release an additional vehicle whenever the expected delay associated with attempting a merge into a detected gap is less than the expected delay associated with waiting for the ramp to clear. Here the controller is a sequential decision-maker that evaluates the expected delay associated with all previously released vehicles that have not yet merged and reflects this information into the control process. Fixed-time and demand-capacity metering controllers are special cases of the control system analysis presented.

Several freeway control projects are presently under way in the United States, and each is attempting to improve the operational characteristics of a freeway by use of an entrance ramp control system. However, the operational modes for the respective control systems and the rationale that underlies individual control system designs appear to be quite different. As a result of this diversity, there appears at first to be no clear-cut design philosophy with which to approach a new access ramp control system design problem. Because of this deficiency, a study group at the Polytechnic Institute of Brooklyn undertook the development of a more unified design theory for dynamic control systems for freeway entrance ramps. The study, initiated in August 1967, was part of the Gulf Freeway Surveillance and Control Project conducted by Texas A&M University in Houston, Texas, for the U.S. Bureau of Public Roads. The portion of that study described in this paper concerns the design of a dynamic control algorithm for an arbitrary entrance ramp configuration, subject to the assumption that control is exercised by use of a green-amber-red traffic signal located at a fixed position on the ramp. Control systems designs that utilize other controller configurations, such as speed signs or multiple signal stations, are not included in this presentation.

For the type of systems considered, 2 identifiable primary functions are (a) to improve the merge service offered to vehicles that enter the freeway and (b) to improve the operation of the freeway. The first function is performed by any system that better coordinates the arrival of the merge vehicle at the merge zone with the availability of a high-quality merge opportunity, and the second function is performed by any system that improves the freeway operational characteristics such as volume, speed, density,
accident rate, and occupancy. These 2 functions are not always compatible, but there is evidence to indicate that compatible solutions do exist. In particular, observation of the Gulf Freeway reveals that operation under conditions of ramp control (6) yields the following:

1. The rush period volume increased by about 10 percent;
2. The speed on the test section increased by about 30 percent;
3. The average travel time over a 5-mile section decreased from 16 to 11 minutes; and
4. The number of accidents during the 2-hour morning peak decreased from 145 to 75 per year for the 3 inbound lanes over the 6.5-mile controlled section.

**DYNAMIC RAMP CONTROL**

To understand the nature of the control system design problem for a freeway entrance ramp required that initial consideration be focused on the Gap Acceptance Control System. In this system any intervehicular headway in the outside freeway lane in excess of \( T_1 \) seconds is defined as a gap into which a vehicle that seeks to enter the freeway may be placed. To achieve the placement of entering vehicles into detected gaps, however, the gap detection process must be carried on sufficiently upstream from the merge point to allow both for synchronization after suitable travel time of vehicles on the ramp and for survival of detected gaps during the passage from the detection point to the merge point. In addition, the gap threshold \( T_1 \) must be sufficiently large so that an acceptable portion of the drivers execute a synchronized moving merge. Within this framework the design problem is then to select locations for the traffic control signal and the gap detector equipment and to specify the gap threshold \( T_1 \).

Location of the control signal is restricted by the need for sufficient room to accelerate prior to the merge maneuver and by the desire to provide sufficient room for the queue that develops behind the signal. Coordinated with this is the need to place the gap detector as close to the merge point as possible to obtain the best possible gap survival conditions. The choice at any particular ramp is thus limited. The specification of \( T_1 \) to provide an acceptable rate of gap acceptance by merging drivers, however, is arbitrary and a large range for choice exists. When \( T_1 \) is chosen to be small, there are many gaps and the smaller of these are likely to be rejected by the drivers. For larger values of \( T_1 \), there are fewer gaps, but the smaller of the gaps are deleted and the likelihood that a driver will reject the offered gap is reduced.

If the design concept is to be properly established, first consideration must be given to the rate at which gaps (i.e., headways in excess of \( T_1 \)) appear in a traffic stream of volume \( q \). Figure 1 shows that the gap detection rate, \( \mu(T_1, q) \), is a function of both the threshold limit \( T_1 \) and the lane volume \( q \). Then, it is necessary to note that for very large gaps the probability that a driver accepts the gap for a moving merge is approximately 1, and for sequentially smaller gaps the probability of acceptance by the driver is reduced toward a finite limit, \( K \), between 0 and 1 (Fig. 2). This lower limit corresponds to the probability that a driver can force entrance into a stream at a point at which no headway was detected. Based on these 2 concepts, subdivision of all gaps into 2 groups is then possible; the 2 groups include those that are accepted for a moving merge and those that are not accepted for a moving
merge (Fig. 3). Corresponding to this division, \( \mu_A(T_1, q) \) is the rate at which accepted gaps are detected, and \( \mu_R(T_1, q) \) is the rate at which rejected gaps are detected; the sum of these quantities, \( \mu(T_1, q) \) is the rate at which all gaps in excess of \( T_1 \) are detected, that is,

\[
\mu(T_1, q) = \mu_R(T_1, q) + \mu_A(T_1, q) \tag{1}
\]

Single-Vehicle-Release-After-Completion Mode

For any specified value of gap threshold, \( T_1 = T \), the rate at which gaps are detected is simply \( \mu(T, q) \). Thus a driver who randomly arrives at the controller suffers an expected delay of \( 1/\mu(T, q) \) seconds before a gap is detected. Then, the driver must proceed down the ramp for which the expected travel time is denoted by \( t_r \). In addition, the probability that the offered gap is unacceptable is equal to the ratio of unacceptable gaps to total gaps, \( \mu_R(T, q)/\mu(T, q) \).

Hence, an extra expected delay associated with the rejection of an offered gap is

\[
\frac{1}{\mu_s(q)}
\]

where \( \mu_S(q) \) is the occurrence rate of gaps that are acceptable for merge by drivers who have previously rejected the gap offered to them and are stopped in the merge zone. These quantities can be used to evaluate the total expected service time for a driver who arrives at the ramp control signal for merge service onto the freeway as

\[
t_e = \frac{1}{\mu(T, q)} + t_r + \frac{\mu_R(T, q)}{\mu(T, q)} \cdot \frac{1}{\mu_s(q)} \tag{2}
\]

As a result, when operation is restricted to a one-at-a-time mode and release is predicated on the completion of the merge maneuver by all previously released vehicles, a single-vehicle-release-after-completion system exists for which the ramp service rate \( \mu' \) is a function of the threshold limit \( T \) and the volume \( q \) in the outside freeway lane. This service rate is

\[
\mu' = \frac{1}{t_e} = \frac{\mu_S(q) \mu(T, q)}{\mu_R(T, q) + \mu_S(q) \left[ 1 + t_r \mu(T, q) \right]} \tag{3}
\]

when \( t_e \) is large compared with \( 1/q \) so that the time interval between the instants at which sequential gaps are sought is large compared with average headway in the stream. (Note that the condition probability of a gap at the instant \( t = T_0 \) given a gap at the instant \( t = 0 \) approaches the unconditioned probability of a gap at the instant \( t = T \) when \( T_0 \) is made very large. For an Erlang headway process on the freeway, this is equivalent to \( t_e \gg 1/q \).)

The Design of a Controller for the Single-Vehicle-Release Mode

One useful and fairly common model for an urban freeway employs stochastic processes with slowly varying parameters to account for both ramp arrivals and highway flows. In particular, the peak-period ramp-arrival process is often described as a time-dependent Poisson process \( (1) \) with \( \lambda(t) \) as shown in Figure 4. In addition, the
Intervehicular spacings for vehicles on the highway are described as independent samples from an Erlang distribution (2, ch. 9):

$$f(t) = \frac{(aq)^a t^{a-1} e^{-aqt}}{(a-1)!}$$

where $a$ is an integer and $q$ varies slowly as a function of time corresponding to average volume (Fig. 5). Based on these descriptions and subject to the assumption that the process parameters vary slowly, one controller design philosophy is to maximize the service rate $\mu'$ subject to a limitation of downstream freeway capacity. The service rate $\mu'$ can be rewritten with the arguments omitted as

$$\mu' = \frac{1}{t_T + (\mu_R + \mu_S)/\mu_S} = \frac{1}{t_T + 1/\mu'}$$

From this form it is seen that, for a given $t_T$, the maxima of $\mu'$ correspond to the maxima of $\mu''$. Hence, setting $d\mu'/dT = 0$ located the values of $T$ corresponding to the relative minima of $\mu'$. In particular,

$$d\mu'/dT = \frac{\mu_S(\mu_R + \mu_S) (d\mu_R/dT) - \mu_S \mu (d\mu_R/dT)}{(\mu_R + \mu_S)^2} = 0$$

reduces to

$$(\mu_R + \mu_S) (d\mu/dT) = (d\mu_R/dT) \mu$$

because $(\mu_R + \mu_S)^2$ is positive and finite, and $\mu S \neq 0$. From Eq. 6,

$$(d/dT) [\ln (\mu) - \ln (\mu_R + \mu_S)] = 0$$

Because $T$ corresponds to the threshold that is set on the minimum spacing between vehicles on the freeway into which a moving merge will be attempted, all spacings in excess of this threshold are gaps. For any given threshold limit, the rate at which gaps appear in a stream with volume equal to $q$ vehicles per second is

$$\mu = q \int_T^\infty \frac{(aq)^a t^{a-1} e^{-aqt}}{(a-1)!} dt$$

$$= q e^{-aqT} \sum_{i=0}^{a-1} \frac{(aqT)^i}{i!}$$

When the probability that a presented gap of $t$ is accepted by a driver for a moving merge is described (2, ch. 9) by

$$P_a (t) = 1 - e^{-Kt}$$

then the rate of successful moving merges is

$$\mu_A = q \int_T^\infty (1 - e^{-Kt}) \left[ \frac{(aq)^a t^{a-1} e^{-aqt}}{(a-1)!} \right] dt$$

$$= q \left\{ e^{-aqT} \sum_{i=0}^{a-1} \frac{(aq)^i}{i!} - \frac{(aq)^a}{(aq + K)^a} e^{-(aq+K)T} \sum_{i=0}^{a-1} \frac{(aq + K)^i}{i!} \right\}$$
and the rate of unsuccessful moving merges is

$$\mu_R = q \frac{(aq)^a}{(aq + K)^a} e^{-(aq+K)T} \frac{(aq + K)T}{\sum_{n=0}^{a-1} \frac{[(aq + K)T]^n}{n!}} \tag{11}$$

When the indicated derivatives are evaluated and employed in Eq. 6, then algebraic manipulations yield

$$\mu_S = q \frac{e^{-(aq+K)T}}{(aq + K)^a} \left\{ \sum_{n=0}^{a-1} \frac{[(aq + K)^a(aq)^n - (aq + K)^n(aq)^a] T^n}{n!} \right\} \tag{12}$$

as the equation from which the optimum threshold settings are obtained, subject to the constraint $T \geq 0$. Because this equation is of the form $\mu_S = f(T)$ where

$$f(T) = A e^{-BT} \sum_{n=0}^{a-1} C_n T^n \tag{13}$$

and because

$$C_n > 0 \text{ for every } n \tag{14}$$

the sum

$$\sum_{n=0}^{a-1} C_n T^n > 0 \text{ for } T > 0 \tag{15}$$

Thus, $f(T) > 0 \text{ for } T > 0$. In addition, the derivative of the expression $f(T)$ is

$$f'(T) = \frac{d}{dT} \left[ A e^{-BT} \sum_{n=0}^{a-1} C_n T^n \right] \tag{16}$$

where

$$f'(T) = -KA(aq + K)^a \left[ e^{-BT} \sum_{n=0}^{a-1} \frac{(aqT)^n}{n!} \right] \tag{17}$$

Inspection of this quantity reveals that $f'(T)$ is negative for all positive $T$. Therefore, $f(T)$ is monotonic decreasing for $T \geq 0$. On this basis, the conclusion is that a unique optimum solution for the controller threshold $T$ exists. That value is 0 when

$$\mu_S < q \frac{1}{(aq + K)^a} \left[ (aq + K)^a - (aq)^a \right] \tag{18}$$

Otherwise, the value is the positive number $T_{opt}$, obtained as the solution to Eq. 12.

After $T_{opt}$ is evaluated as described, the control policy is specified next. In particular, when the sum of the volume on the freeway and the demand on the ramp does not exceed the volume limitation for the freeway, a threshold of $T_{opt}$ is used for the ramp. This ensures the highest possible service rate for demand on the ramp and results in minimum expected delay and minimum expected queue length. When the sum of the freeway volume and ramp demand exceeds the volume limitation, a threshold of $T_{opt}$ is not acceptable. Instead a threshold $T_c$ must be employed such that the sum of the freeway volume and the served portion of the demand is equal to or less than the volume limitation.
At this point, the heretofore undefined quantities of freeway volume, ramp service, and volume limitation must be considered more exactly. For this purpose it is noted that experience and traffic flow theory (3) indicate that the short-term production of any given point on a freeway cannot exceed some upper bound \( Q \), approximately 2,000 vehicles per hour (vph), without significant risk of breakdown in the flow of traffic. Thus the \( T_A \) minute running average of flow past a critical point must be limited to approximately \( QT_A \) vehicles, depending on the facility. When this short-term average volume is below the specified capacity limit, additional vehicles from the ramp can be admitted into the stream, provided that the running average of the sum remains below the critical value. Thus the average ramp service is limited at most to the difference between the limiting and actual average volumes.

Many solutions are possible to the controller design problem that satisfy this restriction. One such possibility allows the threshold to be set at \( T_{\text{opt}} \), provided that the restriction is met, and inhibits all merging otherwise. By this process all available roadway capacity is used as quickly as possible, and additional waiting vehicles are inserted into gaps as additional room for single vehicles arises. This solution, when associated with the limit condition in which \( T_{\text{opt}} \) equals 0, becomes the capacity adjusted metering system (4, 5).

A second approach to the controller design problem involves the gradual adjustment of the threshold as a function of freeway volume. This technique has the usual advantages associated with smooth variation in controller policy; however, it also has the added limitation, extra delay, associated with smoothing. In particular, the first policy carries the risk of inserting an acceptable averaged number of vehicles into the stream too quickly thus causing breakdown, and the second policy includes the risk of adjusting too slowly thus overloading the stream.

The Multiple-Vehicle-Per-Gap Merge Mode

In the generalized release-after-completion merge mode, a string of vehicles is released to attempt to merge into a suitable single gap after all previously released vehicles have completed the merge operation. To design a controller for this action requires only the specification of the gap threshold \( T_n \) that corresponds to the smallest gap into which a string of \( n \) vehicles may attempt a merge. When this is done, the sequence of numbers \( \{T_n\} \) determines the number of vehicles that may attempt to merge into any given gap.

The subsequent analysis and controller design is simplified by the assumption that the gap threshold is chosen so that the probability that the last vehicle in a released string of \( n \) fails to merge is substantially larger than the probability that 2 or more vehicles in the string balk. Thus, the expected service time \( t_{en} \) for a string of \( n \) vehicles that is released to attempt to merge into a gap in excess of \( T_n \) but less than \( T_{n+1} \) is

\[
t_{en} = \text{the expected travel time for the first vehicle to reach the merge zone} + \text{the sum of the expected intervehicular headways between vehicles in the string} + \text{the expected extra delay due to gap rejection by the nth vehicle}
\]

\[
= t_r + (n - 1) h + (\text{Prob n are released and the nth balks}) (1/\mu_S)
\]

\[
= t_r + (n - 1) h + \frac{(1/\mu_S)}{[\mu(T_n) - \mu(T_{n+1})]}
\]

(19)

where

- \( t_r \) = the expected travel time for 1 vehicle on the ramp,
- \( h \) = the expected intervehicular headway,
- \( \mu_S \) = the rate at which vehicles stopped at the merge zone execute merges into the freeway stream,
- \( \mu(T) \) = the rate at which gaps in excess of the threshold setting of \( T \) appear in the stream, and
- \( \mu_{Rn}(T) \) = the merge rejection rate for the nth vehicle in a string when the release threshold is set at \( T \).
Next, it is noted that the rate at which merge opportunities appear in the stream, $\mu_{Total}$, is

$$\mu_{Total} = \sum_{n=1}^{\infty} n (\text{the rate of merge opportunities for strings of exactly n vehicles})$$

$$= \sum_{n=1}^{\infty} n[\mu(T_n) - \mu(T_{n+1})] = \sum_{n=1}^{\infty} \mu(T_n)$$

(20)

Based on this, the expected service time associated with the merge opportunities offered by the stream $T_e$ is

$$T_e = \sum_{n=1}^{\infty} \left[ \frac{\text{(merge opportunity rate for strings of n vehicles) (total expected delay for the merge of a string of n vehicles)}}{\text{(total merger opportunity rate)}} \right]$$

$$= \sum_{n=1}^{\infty} \frac{\mu(T_n) - \mu(T_{n+1})}{\mu_{Total}} \left\{ \frac{1}{\mu(T_1)} t_r + (n-1)h + \left( \frac{1}{\mu_S} \frac{\mu_R(T_n) - \mu_R(T_{n+1})}{\mu(T_n) - \mu(T_{n+1})} \right) + \frac{1}{\mu(T_1)} \right\}$$

$$= \frac{1}{\mu_{Total}} \left\{ \frac{(t_r - h)\mu(T_1) + h\mu_{Total} + \left( \frac{1}{\mu_S} \sum_{n=1}^{\infty} [\mu_R(T_n) - \mu_R(T_{n+1})] \right) + 1}{\sum_{n=1}^{\infty} [\mu_R(T_n) - \mu_R(T_{n+1})] + 1} \right\}$$

$$= h + \frac{\sum_{n=1}^{\infty} \mu_R(T_n) + \mu_S [1 + (t_r - h)\mu(T_1)]}{\mu_S \mu_{Total}}$$

(21)

Finally, $\mu_R(T_n) >> \mu_R(T_{n+1})$ is implied by the assumption that the probability of gap rejection by the last released vehicle in a string is much greater than the probability of a balk by any other vehicle in that string. Thus,

$$T_e = h + \frac{\sum_{n=1}^{\infty} \mu_R(T_n) + \mu_S [1 + (t_r - h)\mu(T_1)]}{\mu_S \mu_{Total}}$$

(22)

and the merge capacity limit for the stream is

$$\mu_{Stream} = \frac{1}{T_e} = \frac{1}{h + \sum_{n=1}^{\infty} \mu_R(T_n) + \mu_S [1 + (t_r - h)\mu(T_1)]}$$

$$= \frac{1}{(h + t')}$$

(23)

where

$$t' = \frac{\mu_S \mu_{Total}}{\sum_{n=1}^{\infty} \mu_R(T_n) + \mu_S [1 + (t_r - h)\mu(T_1)]}$$

(24)

Hence, the merge service capacity is maximized when $t'$ is minimized. A controller design based on this is considered next.
The Design of a Controller for String-Release Mode

The extension of controller action to include the release of strings of vehicles for insertion into adequately large gaps offers several advantages at the expense of separate, individualized merge service. In particular, the safety level associated with the single-vehicle-release-after-merge mode is exchanged for a higher merge capacity when demand indicates that this is necessary. The result is a control law that offers individual service when demand is low and that allows for extended merge capability when demand is high, provided that adequate freeway capacity exists.

Inspection of the expression for the capacity of the ramp controller, $\mu_{\text{Stream}}$, reveals this quantity depends on the set of threshold settings $\{T_n\}$. In general, this expression need not be unimodal, and the coordinates of the local maxima are not always identified by a sequential search. There is a good basis, however, for setting the thresholds by a sequential optimization. In particular, because there is no information included in the model to indicate the actual demand for service, an increase in the ramp capacity by the increase of the 2-vehicle merge rate at the expense of the 1-vehicle merge rate is detrimental to system performance when the actual demand requires only 1-vehicle merges. This situation is contrasted by the following: When the value of $T_1$ is selected to maximize the 1-vehicle merge process and then $T_2$ is selected to maximize the 2-vehicle merge process given $T_1$, the resultant system always provides the highest 1-vehicle merge rate and yields extra capacity through 2-vehicle merges as required by demand. Although this controller may not provide as large a total ramp capacity as is available by the simultaneous selection of $T_1$ and $T_2$, only the actual demand will determine under which controller the ramp provides better service.

When the intervehicular headways on the freeway are described as independent samples from the Erlang distribution

$$f(t) = \left(\frac{aq}{a-1}\right)^{a-1} e^{-aqt}$$

and the probability that the last vehicle in a string of $n$ vehicles accepts an offered gap of $t$ seconds or less is described by the cumulative Erlang

$$P_{an}(t) = 1 - e^{-sn^n t} \sum_{m=0}^{s^n-1} \frac{(sn^n t)^m}{m!}$$

the values of $\mu_{Rn}(T_n)$, $\mu(T_n)$, and $\mu_S$ can be evaluated. Here

$$\mu(T_n) = q \int_{T_n}^{\infty} f(t)dt = q e^{-aqT_n} \sum_{x=0}^{a-1} \frac{(aqT_n)^x}{x!}$$

and

$$\mu_{Rn}(T_n) = q \int_{T_n}^{\infty} [1 - P_{an}(t)] f(t)dt = q \sum_{m=0}^{sn^n-1} \left[ \frac{(sn^n)^m}{m!} \frac{(aq)^a}{(a-1)!} \frac{(a+m-1)!}{(aq+sn^n)^a+m} \right]$$

$$\left[ e^{-(aq+sn^n)T_n} \sum_{x=0}^{m+1} \frac{(aq+sn^n)^x}{x!} T_n^x \right]$$

Likewise, when $P_s(t)$ is the standing merge headway acceptance probability,
\[ \mu_S = q \int_0^\infty P_S(t) f(t) dt \]

and

\[ \mu_S = q \sum_{m=0}^{r-1} \frac{(a + m - 1)!}{m! (a - 1)!} \left( \frac{rc}{qa} \right)^m \]

when \( P_S(t) \) is cumulative Erlang type \( r \) with mean equal to \( 1/C \).

The expression for ramp capacity for this case has been evaluated for some examples subject to the philosophy of sequential optimization. Capacity versus threshold is shown for outside lane volumes between 800 and 2,000 vehicles per hour in Figures 6 through 13 to indicate the system sensitivity to adjustment of the threshold \( T \). Figures 14 through 21 show capacity versus string length for similar volumes. In all cases the lower order thresholds are set at their optimum value before the threshold of interest is varied. In addition, it is assumed that the gap acceptance probability that corresponds to the end vehicle in a string is type 3 Erlang with a mean that varies linearly with string length.

Finally, it is noted that when the maximum string length is restricted to 1, and the gap acceptance probability for single vehicles is Erlang type 1, the string merge capacity simplifies into the 1-vehicle merge capacity previously developed.

CAPACITY AND SENSITIVITY OF A STRING-RELEASE-AFTER-COMPLETION MERGE-CONTROLLER—AN EXAMPLE

At this point the capacity of a ramp control system that releases a string of vehicles to attempt a merge into the freeway stream is considered. For this operational mode the number of vehicles in the string is dependent on the size of the detected gap, and release is conditional on the completion of successful merges by all previously released vehicles. Hence, the ramp is clear before an additional string is released.

Several cases considered for the controller design presented in the previous section included the following:

1. Two alternate length entrance ramps were measured in "seconds of time between the instant the control signal is turned to green and the instant the first merge-vehicle arrives at the merge zone"; thus \( t_r = 5 \) seconds and \( t_r = 10 \) seconds are used to investigate the effect of ramp length.
2. Four alternate gap acceptance criteria were employed to describe drivers that balk at the moving-merge attempt. Here \( 1/c \) equal to 1.5, 3.0, 4.5, and 6.0 seconds are respectively used for the mean gap sizes that drivers who stop at the merge zone find acceptable for a standing merge.
3. Five freeway volumes that correspond to \( q \) equal to 800, 1,100, 1,400, 1,700, and 2,000 vph were investigated to study the effects of variation in volume.

In addition, the following parameter values were employed as reasonable or typical or both:

1. The standing-merge gap acceptance probability is described as Erlang type 3, thus \( r = 3 \);
2. The moving-merge gap acceptance probability for the \( n \)th vehicle in a string is Erlang type 3, thus \( s_n = 3 \) for all \( n \);
3. The mean acceptable gap for the \( n \)th vehicle in a string is \( 1/b_n = 3n - 1.5 \), thus the first, second, third, and \( n \)th vehicles accept a gap of 1.5, 4.5, 7.5, and \( 3n - 1.5 \) seconds respectively on one-half of the merge attempts; and
4. The freeway intervehicular headway process is described by an Erlang type a distribution with mean equal to \( 1/q \) (where \( q \) is the volume), and here, \( a \) is the nearest integer value of the expression \( a = 0.92 e^{0.92} \) and is based on the experimental results obtained on the Gulf Freeway (2).
Figure 6. Single-vehicle-release-after-completion mode with expected ramp travel time of 10 seconds, mean acceptable gap for moving merge of 1.5 seconds, and mean acceptable gap for standing merge of 1.5 seconds.

Figure 7. Single-vehicle-release-after-completion mode with expected ramp travel time of 10 seconds, mean acceptable gap for moving merge of 1.5 seconds, and mean acceptable gap for standing merge of 3.0 seconds.

Figure 8. Single-vehicle-release-after-completion mode with expected ramp travel time of 10 seconds, mean acceptable gap for moving merge of 1.5 seconds, and mean acceptable gap for standing merge of 4.5 seconds.

Figure 9. Single-vehicle-release-after-completion mode with expected ramp travel time of 10 seconds, mean acceptable gap for moving merge of 1.5 seconds, and mean acceptable gap for standing merge of 6.0 seconds.
Figure 10. Single-vehicle-release-after-completion mode with expected ramp travel time of 5 seconds, mean acceptable gap for moving merge of 1.5 seconds, and mean acceptable gap for standing merge of 1.5 seconds.

Figure 11. Single-vehicle-release-after-completion mode with expected ramp travel time of 5 seconds, mean acceptable gap for moving merge of 1.5 seconds, and mean acceptable gap for standing merge of 3.0 seconds.

Figure 12. Single-vehicle-release-after-completion mode with expected ramp travel time of 5 seconds, mean acceptable gap for moving merge of 1.5 seconds, and mean acceptable gap for standing merge of 4.5 seconds.

Figure 13. Single-vehicle-release-after-completion mode with expected ramp travel time of 5 seconds, mean acceptable gap for moving merge of 1.5 seconds, and mean acceptable gap for standing merge of 6.0 seconds.
Before the results of the numerical analysis are discussed, 2 points are worth noting:

1. When the time required by a vehicle to reach the merge zone is $t_r$ seconds (after the signal is changed to green), then the maximum ramp volume in the single-vehicle-release-after-completion mode is $1/t_r$. This quantity equals 720 and 360 vph when the respective values of $t_r$ are 5 and 10 seconds. When strings of $n$ vehicles each are released in the release-after-completion mode and the assumption is used that intervehicular headway is $n$ equal to 3 seconds, the corresponding merge rate is $1/(t_r + 3n - 3)$. 

Figure 14. Multiple-vehicle-release-after-completion mode with expected ramp travel time of 10 seconds, mean acceptable gap for moving merge for $n$th vehicle of 3n - 1.5 seconds, and mean acceptable gap for standing merge of 1.5 seconds.

Figure 15. Multiple-vehicle-release-after-completion mode with expected ramp travel time of 10 seconds, mean acceptable gap for moving merge for $n$th vehicle of 3n - 1.5 seconds, and mean acceptable gap for standing merge of 3.0 seconds.

Figure 16. Multiple-vehicle-release-after-completion mode with expected ramp travel time of 10 seconds, mean acceptable gap for moving merge for $n$th vehicle of 3n - 1.5 seconds, and mean acceptable gap for standing merge of 4.5 seconds.

Figure 17. Multiple-vehicle-release-after-completion mode with expected ramp travel time of 10 seconds, mean acceptable gap for moving merge for $n$th vehicle of 3n - 1.5 seconds, and mean acceptable gap for standing merge of 6.0 seconds.
Figure 18. Multiple-vehicle-release-after-completion mode with expected ramp travel time of 5 seconds, mean acceptable gap for moving merge for $n$th vehicle of $3n - 1.5$ seconds, and mean acceptable gap for standing merge of 1.5 seconds.

Figure 19. Multiple-vehicle-release-after-completion mode with expected ramp travel time of 5 seconds, mean acceptable gap for moving merge for $n$th vehicle of $3n - 1.5$ seconds, and mean acceptable gap for standing merge of 3.0 seconds.

Figure 20. Multiple-vehicle-release-after-completion mode with expected ramp travel time of 5 seconds, mean acceptable gap for moving merge for $n$th vehicle of $3n - 1.5$ seconds, and mean acceptable gap for standing merge of 4.5 seconds.

Figure 21. Multiple-vehicle-release-after-completion mode with expected ramp travel time of 5 seconds, mean acceptable gap for moving merge for $n$th vehicle of $3n - 1.5$ seconds, and mean acceptable gap for standing merge of 6.0 seconds.
As \( n \) approaches infinity, this quantity approaches 1,200 vph for all values of \( t_r \). This limit corresponds to the uncontrolled capacity limit of the entrance ramp.

2. The capacity of an entrance ramp in the release-after-completion mode depends on both the expected length and the expected frequency of the strings of vehicles that may be released to attempt a moving-merge into the freeway stream. For such operation, an increase in the freeway volume always results in a decrease in the number of vehicles in a released string. The expected number of released strings (i.e., the expected frequency), however, may either increase or decrease with an increase in freeway volume because this quantity depends on both the number and the suitability of freeway gaps. In particular, for low freeway volumes almost all gaps are suitable for a merge attempt by 1 vehicle, and an increase in the number of gaps implies an increase in the allowable number of released strings; however, for high freeway volumes only a portion of the gaps are suitable for merge attempts, and an increase in the number of gaps implies both that the average gap becomes smaller and that the number of gaps suitable for merge attempts decreases.

The use of parameter values previously listed in the models that were developed in the prior section makes it possible to numerically evaluate representative entrance ramp capacities. The results of such numerical evaluation with a string-release-after-completion merge controller are presented graphically for ease of interpretation. In Figures 6 through 13, the ramp capacity is shown as a function of threshold setting for the single-vehicle-release-after-completion mode, with volume as a parameter. These curves show that a threshold of approximately 1 second yields a generally near-maximum capacity for all cases. However, the sensitivity of capacity to threshold setting varies substantially. The short ramps (i.e., \( t_r = 5 \) seconds) tend to have higher capacities and greater sensitivity; the \( t_r = 10 \) second cases have lower rates and are less sensitive to variations in threshold. Likewise, there is a functional dependence of ramp capacity on the nature of the merge zone. For short ramps, the ease of execution of standing merges substantially affects the ramp capacity, but on long ramps this quantity is less significant.

More exactly, the numerically evaluated results indicate the following:

1. As the threshold is increased the occurrence of balks becomes less significant, and the capacities are asymptotically identical (i.e., independent of \( c \) for a given \( q \) and \( t_r \)).
2. The quality of service is improved at the expense of added delay when the threshold is increased, and the expected service time for a given threshold \( T_e \) is simply the reciprocal of the associated ramp capacity.
3. For all cases investigated, the largest ramp capacity for a single-vehicle-release-after-completion controller corresponds to a short ramp with an excellent merge zone (i.e., \( t_r = 5 \) and \( 1/c = 1.5 \) seconds). Here, a ramp capacity of approximately 450 vph is indicated for a heavy freeway flow condition (i.e., \( q \) of 1,700 to 2,000 vph), but actual service must be suitably restricted below this value to prevent a breakdown of the stream.
4. The differences between the ramp capacities shown by the curves in Figures 6 through 13 and the maximum capacities of 360 and 720 vph that correspond to \( t_r = 10 \) and 5 seconds respectively are attributed to the combined effects of limiting by the threshold setting and by gap rejection (i.e., by both moving and stopped vehicles).

The maximum ramp capacity available in the string-release-after-completion mode has been evaluated by a sequential search, subject to the restriction that a second vehicle is not released unless a gap of 3 seconds or larger is detected (i.e., because the headway \( h = 3 \) seconds). Similarly, the release of a third vehicle required a gap of at least 6 seconds, the release of a fourth vehicle required a minimum gap of 9 seconds, and so on. The resultant ramp capacities are shown as a function of maximum string lengths in Figures 14 through 21 with freeway volume as a parameter and for various values of \( t_r \) and \( c \). These curves show that the largest benefit of multiple vehicle release occurs at low freeway volumes; at high volumes, little extra capacity is gained. Hence, the largest volume available for the cases investigated is approximately 480 vph when \( t_r = 5 \) seconds, \( 1/c = 1.5 \) seconds, and \( q = 800 \) vph. This is substantially below the limit of 1,200 vph that corresponds to \( q = 0 \). However, when the case of \( t_r = 10 \) seconds and \( 1/c = 3 \) seconds is considered as an example, it is observed that, when the freeway volume gradually drops from 2,000 to 800 vph, the single-vehicle-merge ramp
capacity remains almost constant at approximately 250 vph and the string-release mode yields an automatic increase in capacity up to 350 vph. Finally, it is noted that, when a ramp capacity of 400 vph is required for this ramp, a reduction in $t_R$ from 10 seconds to 5 seconds yields a string-release capacity in excess of 400 vph. However, when a ramp capacity of 500 vph is needed, use of the string-release-after-completion mode is unacceptable and release-before-completion must be implemented. This mode of operation is considered in the next section.

One final point remaining with regard to the numerical results is that the parameter values were chosen on the basis of reasonableness in order to yield typical results for presentation. The models developed are quite general and may be applied to alternate situations by the proper selection of parameter values.

THE RELEASE-BEFORE-COMPLETION MODE

Although the release-after-completion mode exhibits many desirable attributes, such as high safety level and high quality of merge service, the question of whether to release an additional vehicle while previously released vehicles remain on the ramp requires consideration. In particular, when a possible merge opportunity arises for a vehicle awaiting service behind the ramp control signal, the expected service delay for that vehicle may be significantly reduced when that vehicle is released without awaiting merge-completion for the previously released vehicle. However, when this operational mode is to be employed, the question of what value of gap threshold to use must be reexamined.

When a possible merge opportunity is detected in the freeway traffic stream before all previously released vehicles have merged, the question of whether to release the next queued vehicle can be converted into a question of which action minimizes the expected delay. Here the expected delay subject to release of the vehicle must be compared with the expected delay subject to nonrelease to determine which decision to implement. The resultant analysis yields a controller that is a sequential decision-maker in which both the target gap sizes and present state of vehicles on the ramp affect the decision process.

Analysis of the operation of an entrance ramp in the release-before-completion mode can take several forms. One possibility is to assume that the actual ramp service rate is restricted to a value that does not cause a breakdown in the flow of vehicles on the freeway. Then, an investigation of expected delay for the next vehicle awaiting service simplifies to a question of which decision yields the earliest expected merge instant. For this case, the following points are noted:

1. There are $n$ unmerged vehicles on the ramp;
2. The last previous release of a vehicle occurred at $t = t_2$;
3. The target gap sizes are $\tau_1, \tau_2, \ldots, \tau_n$ respectively;
4. A target gap of $\tau_{n+1}$ is detected at $T = t_d$;
5. The expected delay between the completion of standing merges is $1/\mu_S$;
6. The probability that the next vehicle can complete a moving merge into the prospective target is $\prod_{i=1}^{n+1} P_a(\tau_i)$; and
7. The expected delay before a successful merge, given the merge zone is cleared, is $t_R + 1/\mu''$ (i.e., $\mu''$ is the merge rate when $t_R = 0$).

Based on these definitions, the expected additional time required to complete the merge of the $(n + 1)$th vehicle (i.e., beyond the time required to complete the merge of the $n$th vehicle and provided that no balk has occurred) is

$$\Delta t_y = \left[1 - \prod_{i=1}^{n+1} P_a(\tau_i)\right] \left(1/\mu_S\right) + \left[\prod_{i=1}^{n} P_a(\tau_i)\right] [t_d - t_L]$$

when the vehicle is released to attempt entry into the $\tau_{n+1}$ gap, and
\[ \Delta t_n = t_r + 1/\mu'' \]  

(31)

when release is inhibited until the ramp is clear.

Comparison of these 2 quantities reveals that when the gap \( r_{n+1} \) satisfies the inequality

\[ P_a(r_{n+1}) = [(t_d - t_c)\mu_S - 1] \prod_{i=1}^{n} P_a(\tau_i) - [\mu g + (\mu S/\mu'') - 1] \]

\[ > f[\mu_S, \mu'', t_r, \tau_1, \tau_2, \ldots, \tau_n] \]

(32)

the \((n + 1)\)th vehicle suffers less delay in the merge process by not waiting for the ramp to clear. Thus for the operational condition in which the function \( f(\cdot) \) on the right side of the inequality is negative, the inequality is always satisfied, and any value of \( r_{n+1} \) is acceptable for a merge attempt. Similarly, when \( f(\cdot) \) exceeds unity, no gap is acceptable because \( P_a(r_{n+1}) \) is a gap-acceptance probability and cannot exceed unity. Finally, when \( f(\cdot) \) is between 0 and 1, a threshold \( T_{n+1} \) exists such that

\[ T_{n+1} = P_a^{-1}[f(\cdot)] \]

(33)

where \( P_a^{-1}(\cdot) \) is the inverse of the gap acceptance probability function and \( P_a^{-1}[P_a(t)] = t \).

For a ramp that has experienced a stoppage and offers zero probability of a moving merge for the \((n + 1)\)th vehicle, the expected delays associated with the release or the nonrelease of that vehicle at \( t_d \) are respectively

\[ \Delta t'_r = 1/\mu_S \]

(34)

and

\[ \Delta t'_n = t_r + 1/\mu \]

(35)

Comparison of these quantities indicates that, for any ramp on which the expected travel time \( t_r \) exceeds the difference between the expected delay \( 1/\mu_S \) (associated with a standing merge from the merge zone) and the travel-free service time \( 1/\mu'' \) (associated with the successful clear-ramp, single-vehicle-merge rate), the release of the next vehicle into any size gap is warranted. Setting the probability of a moving merge for the \((n + 1)\)th vehicle in the previous case equal to 0 reinforces this conclusion; this is exactly the same criterion as that for release emerges.

The Design of a Controller for the Release-Before-Completion Mode

Under those operational conditions in which an increase becomes necessary or desirable in the production of a particular entrance ramp above the level achieved in the release-after-completion mode, it can be achieved by operation in the release-before-completion mode. To do so, however, involves a trade-off between the quality of merge-service offered and the merge production.

For a long ramp (i.e., \( t_r \) is large) with a merge zone that is unobstructed so that the expected standing merge delay is comparable to (though more than) the expected delay in the attempt of a moving merge, analysis has shown that the release-before-completion mode offers an improved merge rate for the ramp, provided that the freeway stream does not break down. This property allows for a variation of the controller threshold (or merge-rate limit or both) to increase the ramp production as required within the limits necessary to preserve the stability of the stream. Such operation has been implemented on the Gulf Freeway when queue lengths beyond the available storage capacity have made action necessary. In addition, ramp operation with a fixed threshold in the release-before-completion mode has been more the rule than the exception on that Freeway.

By contrast, when a ramp is short (i.e., \( t_r \) is small) and the merge zone is marginal so that standing merge opportunities are rare compared with moving merge opportunities,
operation in the release-before-completion mode does not always yield an increase in ramp merge capacity. In particular, analysis reveals that when

\[ t_r < \left( \frac{1}{\mu_g} \right) - \left( \frac{1}{\mu'} \right) \]  

(36)
a stoppage of a released vehicle warrants that the release-before-completion operation be inhibited until the ramp is cleared. The existence of this restriction in the release-before-completion operation, however, is temporary and in no way negates the overall advantage of increased ramp capacity that results, provided that the controller selects only adequately large gaps that occur within an adequately short period after the last previous release.

**SUMMARY**

Under either control philosophy described, the threshold that yields the maximum ramp capacity must be identified first and, based on that value, a threshold setting is established that both provides an appropriate level of merge service and preserves the stability of the freeway stream against breakdown in flow caused by excess density. By this process, merge capacity will often be reduced below its maximum value to increase the level of service offered to drivers. Thus, the controller may attempt to operate in a mode in which a single vehicle is released whenever the ramp is clear, the demand is low, and the probability of a successful moving merge is adequately high. When it is necessary to increase the available ramp merge capacity, the operational mode is modified so as to reduce the level of service by the relaxation of control limits until adequate merge capacity is achieved. In this way the controller is continuously adjustable to both actual demand and actual freeway volume.

Based on analysis of the results obtained by use of the control modes developed in this paper, we believe a sound analytic theory exists for the design of freeway entrance ramp control systems. In particular, the relationship between merge capacity and system parameters is explicitly developed for several control modes. These results make possible the evaluation of both the capacity limitations inherent in the design of a particular ramp control system and the control system parameters necessary to provide a specified merge capacity for the ramp.

To aid in the design process, several typical ramp control configurations were analyzed in detail. These included the following conditions: (a) placement of a traffic control signal at between 5 and 10 seconds of travel time from the merge zone, (b) mean gap acceptance limits for vehicles stopped in merge zone of between 1.5 and 6.0 seconds, and (c) freeway volumes in the outside lane of between 600 and 2,000 vph. As a result of this analysis, the following observations are made:

1. The gap rejection phenomenon was found to increase the minimum expected ramp service time, in the single-vehicle-release-after-completion mode, to a level between 1.25 and 3.0 times the expected vehicular travel time on that ramp. For example, for a long ramp with an expected travel time of 10 seconds, the maximum ramp capacity is reduced to between 50 and 80 percent of the 360-vph limit that the 10-second travel time imposes on this mode; 3,600 seconds per hour divided by a minimum of 10 seconds per vehicle yields a maximum flow in the single-vehicle-release-after-completion mode of 360 vph. Here, the larger capacity corresponds to a volume of 800 vph in the outside freeway lane; the lower capacity occurred with a freeway volume of 2,000. In both cases, the assumption is that a relatively good merge zone exists because the mean acceptable gap for a standing merge from the merge zone was defined to equal 1.5 seconds. For a second example, with an expected travel time of 5 seconds and a short merge zone, the results indicate a reduction in maximum ramp capacity to between 35 and 65 percent of the limit of 720 vph that the 5-second travel time imposes on this mode. Again, capacity depended on freeway volume.

2. The release of more than 1 vehicle per string can increase the ramp capacity without forsaking the release-after-completion operational mode. For such operation, multiple-vehicle-release opportunities are rare when high freeway volumes exist and
little is gained in these instances. However, when freeway volumes are low (e.g., 800 vph per lane), improvements of approximately 50 percent over single-vehicle operation are possible.

3. Operation in the string-release-after-completion mode appears to be limited at most to between 15 and 40 percent of the uncontrolled ramp capacity, with the higher production generally corresponding to the lower freeway volumes (i.e., 800 vph per lane). To provide ramp capacities in excess of this limit requires that release-before-completion operation be employed.

4. When the difference between the expected service time required to complete a standing merge from the merge zone is less than the total expected service time required to complete a moving merge from behind the ramp control signal (i.e., including the expected travel time), then the release-before-completion mode exhibits a maximum ramp capacity that equals the uncontrolled ramp capacity. However, the actual service provided by this controller must be constrained by consideration of freeway stream stability and actual demand characteristics. For example, metering at any fixed rate below 1,200 vph would be possible for several of the cases investigated in this report, provided that stream stability was not a problem.

REFERENCES
This report presents further information on the operation of a motorist-aid telephone system on a rural freeway. Stranded motorists' needs and the ways in which these needs are met are examined. Data from observations made during the summer of 1968 and the winter of 1969 indicate about 50 percent of drivers who needed aid used the motorist-aid telephones. The rate for vehicle stops of 12 min or longer was 1 stop in 26 miles each 66 min in the summer survey and 1 stop each 99 min in the winter survey. This is consistent with other reported findings that show greater assistance needs in summer. Assistance rates may correlate directly to traffic volumes, but these data have not been fully analyzed. The percentage of trucks requiring assistance is consistently large compared to their percentage of the traffic stream. Of those that stopped 12 min or longer, 48 percent were serviced by the truck drivers or the drivers were assisted by passing drivers. Drivers of passenger cars requiring mechanical aid are most likely to phone for aid. A comparison of the data from the control section on US-23 to those from the study section on I-94 shows that trucks on US-23, at least, would have greatly reduced their stopped time had aid phones been available to them. The surveys of motorists passing through the motorist-aid phone area on I-94 showed approximately 87 percent favoring such a system. Seventy-five percent of the users of the aid phones travel the study section once a month or more often and, not surprisingly, 97.4 percent of those who have used the phones think the system should be expanded.

**DESCRIPTION OF MOTORIST-AID TELEPHONE SYSTEM**

A 30-mile study section on I-94, between Jackson and Battle Creek, was selected for the experiment (Figs. 1 and 2). I-94 is a major east-west freeway connecting Detroit to Chicago. About 21 percent of its 14,000 average annual daily traffic is commercial.

The motorist-aid telephone system is entirely state-owned with the exception of the leased lines connecting the freeway circuits to the State Police posts. These lines are

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leased from the Michigan Bell Telephone Company. There are 31 pairs of phones over the 30 miles. The east 17 pairs of phones are spaced at approximately 3,400 ft and the remaining 14 pairs at approximately 5,400 ft. The phones are numbered 1 through 62; phones 1 through 28 are connected to the Jackson State Police post, and phones 29 through 62 are connected to the Battle Creek State Police post.

Each motorist-aid telephone is 13.5 ft from the edge of the pavement. A red weatherproof cabinet containing the handset is attached to the downstream side of a 12-ft aluminum pole with a blue light on top. A blue sign displaying a white telephone symbol is also attached facing traffic (Fig. 3a and b).

All telephone and power cables within the right-of-way are underground. The 30-mile section is signed at the beginning, end, and midpoint (Fig. 3d and e). Mileage markers are placed every two-tenths mile along the study section to identify stopping location (Fig. 3c).

To use the telephones, the motorist opens the cabinet door and lifts the handset from the hook. The dispatcher at the police post is notified by a ring and a red light that
identifies the calling site (Fig. 4). The dispatcher answers the call and obtains information necessary to assist the motorist and completes a questionnaire (Fig. 21). The State Police usually supply gasoline to a motorist with this need. For other needs, the proper commercial agency is contacted to service the motorist.

**SYSTEM OPERATION AND MAINTENANCE**

Since the last interim report, the system has operated well, but there have been some periods when a phone or a circuit was inoperative. Operational problems still occur, ranging from lightning strikes, handsets or the roadside box being vandalized, leased line problems, vehicle damage to underground circuits, and short circuits in the terminal boxes caused by flooding.

Within a 6-month period, 6 to 8 phones or circuits were out of service because of lightning damage. One site was knocked down by a vehicle, 9 handsets or cord damages occurred, and 1 transformer was stolen. This repair activity, plus relamping and placement of a capacitor in each phone site, has kept one man busy full time troubleshooting and repairing. The new capacitor installations have eliminated more false ringing problems, many of which were apparently caused by power interruptions. Recently a building remodeling at the Battle Creek State Police post caused those circuits to be out of service for several days while the display panel was relocated.

Aside from these random problems, the system has been operating well. Weekly checks show that occasionally the State Police personnel are too busy with other duties to answer quickly; however, we have not heard recently that anyone tried to call and could not get an answer. A few returns of an earlier handout questionnaire contained comments of this type. It does appear that we are not receiving data on many of the calls for assistance when in fact assistance to the motorist is provided.

Plans have been made to interface a tape recorder with the system at each post. These recorders will be activated when the handsets at the posts are lifted for an incoming call. This addition should fill the gap in total information on call activity. Normal police post activity is sufficient to take most of the dispatchers' time and, thus, calls on the motorist-aid phones are usually handled as briefly as possible and perhaps at times are not recorded at all.

In any case, the system can still be of value even with some periods when a phone or circuit is temporarily inoperative. The forthcoming period of operation with the tape recorders should provide a broader base of information on the real use of the system.

**MOBILE OBSERVATIONS OF STRANDED MOTORISTS**

Mobile observations were made during the summer of 1968 and winter of 1969 on the study section and the control section on US-23. The 10-mile section on US-23 (Fig. 1) was chosen as a control to provide a comparison of stranded motorist activities on this section of roadway with those on I-94.

The mobile survey was designed to let the stopped motorist take some action before being interviewed. Four survey cars were equally spaced in a 26-mile loop from Parma Road to 11-Mile Road on I-94 (Fig. 2). The time interval between cars was 12 min; by definition, motorists stopped over 12 min were deemed stranded. The first survey car to spot a stopped vehicle reported to the next survey car in the loop by radio. If the
stopped vehicle was still present when the next survey car approached, an interview was taken and recorded on the form shown in Figure 5. A fifth survey car took the stopped survey car’s position in the loop. On the US-23 control section, there was 1 survey car whose driver stopped when he saw a stopped vehicle and waited at a distance for the stopped motorist to take action. When the motorist started remedial action, the survey car driver moved up and interviewed the stopped motorist. Obviously,

**Figure 5.** Interview form used in stranded-vehicle summer and winter surveys.

**TABLE 1**

<table>
<thead>
<tr>
<th>Reason</th>
<th>Passenger Cars</th>
<th>Other Vehiclesa</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Number</td>
<td>Percent</td>
</tr>
<tr>
<td>Tire failure</td>
<td>57</td>
<td>48</td>
</tr>
<tr>
<td>Gas, water, or oil</td>
<td>21</td>
<td>18</td>
</tr>
<tr>
<td>Mechanical, tow</td>
<td>19</td>
<td>16</td>
</tr>
<tr>
<td>Mechanical, no tow</td>
<td>21</td>
<td>18</td>
</tr>
<tr>
<td>Accident</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Fire</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td>119</td>
<td>100</td>
</tr>
</tbody>
</table>

aIncludes 48 trucks, 3 buses, and 2 motorcycles.
TABLE 2
Actions Taken by Stranded Motorists to Obtain Aid on I-94 During 1968 Summer Survey

<table>
<thead>
<tr>
<th>Action</th>
<th>Number</th>
<th>Percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>Off-freeway aid</td>
<td>90</td>
<td>52</td>
</tr>
<tr>
<td>Used aid phonesa</td>
<td>48b</td>
<td>28</td>
</tr>
<tr>
<td>Used public phones</td>
<td>4</td>
<td>2</td>
</tr>
<tr>
<td>Walked</td>
<td>14c</td>
<td>8</td>
</tr>
<tr>
<td>Hitchhiked</td>
<td>18d</td>
<td>11</td>
</tr>
<tr>
<td>Abandoned vehicle for over 10 hours</td>
<td>6</td>
<td>3</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>82</td>
<td>48</td>
</tr>
<tr>
<td>Self-help</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tire</td>
<td>36e</td>
<td></td>
</tr>
<tr>
<td>Mechanical</td>
<td>9f</td>
<td></td>
</tr>
<tr>
<td>Used own radio</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Drove to service</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>Others helped</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Survey group</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>Passerby</td>
<td>11</td>
<td></td>
</tr>
<tr>
<td>Police</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Unknown</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>172</td>
<td>100</td>
</tr>
</tbody>
</table>

aOf those interviewed, 40 motorists or 23 percent were not aware of the aid phones.
b52 percent of the 90 motorists needing off-freeway aid. c11 of these were aware of the aid phones. d15 of these were aware of the aid phones. e49 percent of the motorists with tire needs. f15 percent of the motorists with mechanical needs.

passing motorists willing to give aid. The 26 people who walked or hitchhiked, but were aware of the aid phones, apparently assumed that they could obtain aid either faster, or cheaper, themselves. It is rather puzzling, however, that 77 percent of the needy motorists were aware of the phones but only 52 percent made use of them.

Observations were made on I-94 during the winter for 71 hours over 10 days during which 36 interviews were taken with motorists stopped 12 min or longer. Results are given in Table 3. Although the total number of interviews is smaller, the percentage distribution of the reasons cars (69 percent) and trucks (31 percent) stopped during the winter survey is the same as that of the summer survey. Trucks had a disproportionate share of the breakdowns. The changes in the percentages in Table 3 can be attributed largely to the cooler weather and correspondingly fewer tire failures. Trucks obviously have more mechanical needs in winter; however, gas, water, and oil needs were not evident in winter for trucks. Some of these variations may also be the result of the smaller winter sample. Table 4 gives the actions taken by drivers during the winter survey to meet their various needs when their travel was interrupted. More stopped winter drivers (89 percent) than summer drivers were aware of the aid phones; however, only 50 percent chose to utilize them.

TABLE 3
Reasons for Vehicle Stops on I-94 During 1969 Winter Survey

<table>
<thead>
<tr>
<th>Reason</th>
<th>Passenger Cars</th>
<th>Trucks</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Number</td>
<td>Percent</td>
</tr>
<tr>
<td>Tire failure</td>
<td>6</td>
<td>24</td>
</tr>
<tr>
<td>Gas, water, or oil</td>
<td>4</td>
<td>16</td>
</tr>
<tr>
<td>Mechanical, tow</td>
<td>4</td>
<td>16</td>
</tr>
<tr>
<td>Mechanical, no tow</td>
<td>3</td>
<td>12</td>
</tr>
<tr>
<td>Accident</td>
<td>3</td>
<td>12</td>
</tr>
<tr>
<td>Fire</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>4</td>
<td>16</td>
</tr>
<tr>
<td>Total</td>
<td>25</td>
<td>100</td>
</tr>
</tbody>
</table>
Data from the survey on US-23 are given in Table 5. There are no aid phones on this control section, and 63 interviews or observations of vehicles stopped for 12 min or longer were made during the summer and 35 during the winter. The number obtained on US-23 during the winter is close to the number (36) obtained during the winter survey on I-94 and is difficult to explain because of the method of survey. Although the total number of interviews is small, trucks had a larger share of the needs. They make up 17 percent of the traffic but 29 percent of the summer stops and 51 percent of the winter stops. The major problem for passenger cars in both winter and summer is tire failure, and that for trucks in the summer is mechanical and in the winter, tire failure. Tire problems in winter are rather surprising inasmuch as tire failures are usually reduced in cold weather.

Table 6 gives the actions taken by motorists to obtain aid. The lack of aid phones forced 48 percent of the motorists on US-23 to leave the freeway to obtain aid; only 24 percent of the motorists on I-94 left the freeway to obtain aid. The group leaving the

### Table 4: Actions Taken by Stranded Motorists to Obtain Aid on I-94 during 1970 Winter Survey

<table>
<thead>
<tr>
<th>Action</th>
<th>Number</th>
<th>Percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>Off-freeway aid</td>
<td>7</td>
<td>29</td>
</tr>
<tr>
<td>Used aid phones</td>
<td>5</td>
<td>20</td>
</tr>
<tr>
<td>Other motorists used aid phones</td>
<td>5</td>
<td>20</td>
</tr>
<tr>
<td>Walked</td>
<td>7</td>
<td>28</td>
</tr>
<tr>
<td>Hitchhiked</td>
<td>3</td>
<td>11</td>
</tr>
<tr>
<td>Abandoned vehicle for over 10 hours</td>
<td>4</td>
<td>15</td>
</tr>
<tr>
<td>Self-help</td>
<td>3</td>
<td>11</td>
</tr>
<tr>
<td>Others helped</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>Police</td>
<td>6</td>
<td>22</td>
</tr>
<tr>
<td>Unknown</td>
<td>1</td>
<td>4</td>
</tr>
<tr>
<td>Total</td>
<td>36</td>
<td>100</td>
</tr>
</tbody>
</table>

### Table 5: Reasons for Vehicle Stops on US-23 during Summer and Winter Surveys

<table>
<thead>
<tr>
<th>Reason</th>
<th>Summer</th>
<th>Winter</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number</td>
<td>Percent</td>
<td>Number</td>
</tr>
<tr>
<td>Tire failure</td>
<td>17</td>
<td>39</td>
</tr>
<tr>
<td>Gas, water, or oil</td>
<td>12</td>
<td>26</td>
</tr>
<tr>
<td>Mechanical, tow</td>
<td>5</td>
<td>11</td>
</tr>
<tr>
<td>Mechanical, no tow</td>
<td>5</td>
<td>11</td>
</tr>
<tr>
<td>Accident</td>
<td>2</td>
<td>5</td>
</tr>
<tr>
<td>Stuck off road</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>4</td>
<td>9</td>
</tr>
<tr>
<td>Total</td>
<td>45</td>
<td>100</td>
</tr>
</tbody>
</table>

### Table 6: Actions Taken by Stranded Motorists to Obtain Aid on US-23 during Summer and Winter Surveys

<table>
<thead>
<tr>
<th>Action</th>
<th>Summer Motorists</th>
<th>Winter Motorists</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number</td>
<td>Percent</td>
<td>Number</td>
</tr>
<tr>
<td>--------</td>
<td>---------</td>
<td>--------</td>
</tr>
<tr>
<td>Used public phones</td>
<td>6</td>
<td>10</td>
</tr>
<tr>
<td>Walked</td>
<td>14</td>
<td>22</td>
</tr>
<tr>
<td>Hitchhiked</td>
<td>6</td>
<td>10</td>
</tr>
<tr>
<td>Abandoned vehicle for over 10 hours</td>
<td>4</td>
<td>6</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>33</td>
<td>52</td>
</tr>
<tr>
<td>Self-help</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tire</td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td>Mechanical</td>
<td>8</td>
<td>24</td>
</tr>
<tr>
<td>Drove to service</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Others helped</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Survey group</td>
<td>3</td>
<td>9</td>
</tr>
<tr>
<td>Passerby</td>
<td>9</td>
<td>27</td>
</tr>
<tr>
<td>Police</td>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>Unknown</td>
<td>2</td>
<td>7</td>
</tr>
<tr>
<td>Total</td>
<td>63</td>
<td>100</td>
</tr>
</tbody>
</table>
freeway were considered to be those who used public phone, walked, hitchhiked, and abandoned vehicle. These varying percentages may not be entirely realistic because of the small numbers involved; however, clearly demonstrated is the forced reliance on self-help and help from passing motorists. One-third or more of the stranded motorists received aid from other motorists.

**LEVELS OF SERVICE TO STRANDED MOTORISTS IN TERMS OF AID TIME**

Figure 6 shows an ideal cumulative distribution function for levels of service in which causative factors are related to total time required for the stranded motorists to obtain aid. The first characteristic is that the did-not-use-phone (DNUP) line begins to increase to the left of the used-phone (UP) line. The justification for this is that motorists who are relatively fortunate to be disabled near an intersection with service, or otherwise are able to obtain aid immediately, will realize less delay than most phone users. Thus, there may well be this small privileged class when the system is in the ideal state. The second characteristic is that the UP increased at a rate greater than that of the DNUP. The justification is that the use of the phone should initiate a chain of communications and service links that is reasonably uniform in its capability to aid the stranded motorist. Thus, the spread of time over which aid is given should be less for the phone user than for the individual who does not use the phone. The third characteristic is that the UP obtains 100 percent to the left of the DNUP. The justification is that the use of the phone should ensure that a motorist is not stranded for an extreme length of time. The person who does not use the phone does not have this assurance. In summary, the greater percentages of the DNUP during the excellent level of service times might be anticipated and do not indicate a defect in the system. However, the UP should quickly overcome this advantage and reach 100 percent without long flat periods of time as might also be anticipated for the DNUP. The empirical cumulative distribution functions are shown in Figures 7 through 19. The differences in the number of vehicles shown in these figures and that given in earlier tables is due to the availability of aid-time data.

**Summer Survey on Study Section**

Tire aid was required by 56 stranded passenger cars. Of these, 36 (64 percent) helped themselves and 20 (36 percent) received help from others. Among these 20, 9 (45 percent) used the phones. Figure 7 shows that there is little difference in the aid time of those who used the phones and the aid time of those who did not. More than 80 percent of the group took less than 1 hour to fix their tires.
Figure 7. Aid time for passenger cars with tire failure on I-94 during 1968 summer survey.

Figure 8. Aid time for passenger cars with mechanical failure on I-94 during 1968 summer survey.

Figure 9. Aid time for passenger cars needing water, gas, or oil on I-94 during 1968 summer survey.
Forty passenger cars required mechanical aid. Nine (23 percent) of the drivers helped themselves, and 31 (77 percent) received outside help. Of the 31, 17 (55 percent) used the phones. The cumulative curve shown in Figure 10 is very close to the ideal curve; it shows that a high percentage (77 percent) of the passenger cars with mechanical trouble needed outside help and the phones provided more than half of this assistance.

Nineteen passenger cars required gas, water, or oil (Fig. 9). Four of the drivers used the phones. One of these received help from the survey group in 22 min, but another hitchhiked to get gas and took 62 min. The other 2 who called waited 40 to 66 min to get help from the State Police. The 15 who did not call received help as follows:

<table>
<thead>
<tr>
<th>Time Required, min</th>
</tr>
</thead>
<tbody>
<tr>
<td>Survey group</td>
</tr>
<tr>
<td>Other motorists</td>
</tr>
<tr>
<td>Walked to the nearest gas station</td>
</tr>
<tr>
<td>Walked to rest area for water</td>
</tr>
<tr>
<td>Self-help</td>
</tr>
<tr>
<td>Unknown assistance</td>
</tr>
</tbody>
</table>

Figure 11. Aid time for trucks with mechanical failure on I-94 during 1968 summer survey.
Fourteen of these 15 drivers were aware of the aid phones before they tried to get help. It is obvious that, to obtain this category of services, drivers will do many things rather than use the aid phones, even though they are well aware that the phones are available. The reason for this reluctance can only be guessed because these motorists were not asked this specific question.

Tire aid was required by 16 trucks. The drivers of 5 (31 percent) used the phones and the drivers of 11 (69 percent) did not (Fig. 10). Of these 11, 4 helped themselves, 1 used the pay phone, 1 received help from his own company by using his truck phone, 1 drove to the gas station, and 1 received help from another truck.

Nineteen trucks required mechanical aid. Seven drivers (37 percent) used the phones (Fig. 11). There is little difference in the aid time of those who used the phones and those who did not.

The time distribution of the trucks that required gas, water, or oil is very close to that of the ideal curve (Fig. 12). The sample size, however, is very small.

Winter Survey on Study Section

Drivers of 36 stopped vehicles were interviewed during the winter survey. Of these, 15 used the phones and 21 did not. The aid times for the passenger cars are shown in Figures 13, 14, and 15, and for the trucks in Figures 16 and 17. The sample sizes are small; however, they show promise of being close to the ideal as the sample size increases.

Surveys on Control Section

Figures 18 and 19 show the aid-time distributions for the 3 most common needs of stopped vehicles on the control section on US-23. The short time grouping for the passenger cars apparently was a result of 2 factors: The sample is limited, and the percentage of vehicles on US-23 aided by other motorists was twice that on I-94. Hence, some of these short aid times may well have been 2 or 3 hours under other circumstances.

The stranded-truck activity shows a marked reduction in delay in all categories for I-94 trucks whose drivers used the phones compared to the US-23 drivers (Fig. 19) who had to obtain aid by other means. In both of these distributions, the sample sizes are rather limited.
Figure 15. Aid time for passenger cars needing water, gas, or oil on I-94 during 1969 winter survey.

Figure 16. Aid time for trucks with tire failure on I-94 during 1969 winter survey.

Figure 17. Aid time for trucks with mechanical failure on I-94 during 1969 winter survey.

Figure 18. Aid time for passenger cars on US-23 during summer survey.
ANALYSIS OF HANDOUT QUESTIONNAIRE RESPONSES

In this survey, taken on I-94 in August 1968 and January 1969, a return-mailer questionnaire (Fig. 20) was distributed to approximately 5,000 drivers during each period. About 28 percent were returned; a summary of the responses is given in Table 7. Out-

---

Figure 19. Aid time for trucks on US-23 during summer survey.

Figure 20. Questionnaire handed to stranded motorists on I-94 during summer and winter surveys.
# Table 7

## Summary of Responses of Stopped Vehicle Drivers in Questionnaires Handed to Them

<table>
<thead>
<tr>
<th>Response</th>
<th>Percent of Drivers</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Summer</td>
</tr>
<tr>
<td><strong>Type of vehicle</strong></td>
<td></td>
</tr>
<tr>
<td>Passenger, in county</td>
<td>18.8</td>
</tr>
<tr>
<td>Passenger, in state, out of county</td>
<td>43.5</td>
</tr>
<tr>
<td>Passenger, out of state</td>
<td>26.6</td>
</tr>
<tr>
<td>Panel and pickup cars with trailer, in county</td>
<td>0.3</td>
</tr>
<tr>
<td>Panel and pickup cars with trailer, out of state</td>
<td>1.4</td>
</tr>
<tr>
<td>Truck, single unit</td>
<td>1.4</td>
</tr>
<tr>
<td>Truck, combination</td>
<td>0.3</td>
</tr>
<tr>
<td><strong>Opinion of aid phones</strong></td>
<td></td>
</tr>
<tr>
<td>Necessary service; should be expanded</td>
<td>46.9</td>
</tr>
<tr>
<td>A convenience; would like to see it expanded</td>
<td>40.3</td>
</tr>
<tr>
<td>A convenience; but not necessary</td>
<td>10.3</td>
</tr>
<tr>
<td>Prefer past method of obtaining aid, such as raised hood, flare, or handkerchief on door</td>
<td>0.7</td>
</tr>
<tr>
<td><strong>Frequency of freeway use</strong></td>
<td></td>
</tr>
<tr>
<td>Almost every day</td>
<td>12.7</td>
</tr>
<tr>
<td>Almost every week</td>
<td>18.7</td>
</tr>
<tr>
<td>Almost every month</td>
<td>25.4</td>
</tr>
<tr>
<td>Once or twice a year</td>
<td>28.0</td>
</tr>
<tr>
<td>Less than once a year</td>
<td>15.2</td>
</tr>
<tr>
<td><strong>Trip purpose</strong></td>
<td></td>
</tr>
<tr>
<td>Social-recreational</td>
<td>44.1</td>
</tr>
<tr>
<td>School</td>
<td>2.9</td>
</tr>
<tr>
<td>Shopping</td>
<td>2.3</td>
</tr>
<tr>
<td>Business</td>
<td>6.9</td>
</tr>
<tr>
<td>To or from work</td>
<td>28.8</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>15.0</td>
</tr>
<tr>
<td><strong>Motorist aid phone signing considered inadequate</strong></td>
<td>10.9</td>
</tr>
</tbody>
</table>

*Note: We are conducting a statistical study in this area. You are asked to rate the people we stop by this system. I would like to ask you a few questions. If you don’t wish*

---

**Figure 21.** Interview form used by State Police dispatcher in answering calls on motorist-aid phones.
of-state traffic is higher in summer (30 percent) than in winter (16.3 percent). The trucks show up less than actual percentages because more of the surveys were taken in the daytime than at night, and the relative volumes of trucks to cars increase greatly at night. Both the summer and winter (87 percent) indicated strong favor for the aid phones.

### TABLE 8

<table>
<thead>
<tr>
<th>Location of Call Phone and Aid Requested</th>
<th>Percent of Calls</th>
<th>Location of Call Phone and Aid Requested</th>
<th>Percent of Calls</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td></td>
<td>Oil</td>
<td>2.0</td>
</tr>
<tr>
<td>All phone sites 1 through 34^a</td>
<td>52.0</td>
<td>Mechanical, tow required</td>
<td>19.2</td>
</tr>
<tr>
<td>All phone sites 35 through 62^b</td>
<td>48.0</td>
<td>Mechanical, no tow required</td>
<td>16.8</td>
</tr>
<tr>
<td>Total eastbound phone sites</td>
<td>47.6</td>
<td>Accident, medical aid and tow required</td>
<td>1.1</td>
</tr>
<tr>
<td>Total westbound phone sites</td>
<td>52.4</td>
<td>Accident, medical aid required</td>
<td>1.1</td>
</tr>
<tr>
<td>Eastbound phone sites 2 through 34 (even)</td>
<td>25.0</td>
<td>Accident, tow required and no medical aid</td>
<td>1.3</td>
</tr>
<tr>
<td>Eastbound phone sites 35 through 62 (even)</td>
<td>22.6</td>
<td>Accident, neither medical nor tow required</td>
<td>4.9</td>
</tr>
<tr>
<td>Westbound phone sites 1 through 33 (odd)</td>
<td>26.6</td>
<td>Stuck off road</td>
<td>3.1</td>
</tr>
<tr>
<td>Westbound phone sites 35 through 61 (odd)</td>
<td>25.8</td>
<td>Fire</td>
<td>1.3</td>
</tr>
<tr>
<td>Aid requested (1968)</td>
<td></td>
<td>Police action</td>
<td>2.0</td>
</tr>
<tr>
<td>Tires</td>
<td>22.0</td>
<td>Miscellaneous</td>
<td>0.9</td>
</tr>
<tr>
<td>Gas</td>
<td>21.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Water</td>
<td>4.4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

---

a Average 3,400 ft spacing between pairs.

b Average 5,400 ft spacing between pairs.
TABLE 9
SUMMARY OF RESPONSES OF PHONE USERS IN QUESTIONNAIRES MAILED TO THEM

<table>
<thead>
<tr>
<th>Response</th>
<th>Percent of Users</th>
</tr>
</thead>
<tbody>
<tr>
<td>Frequency of travel on this section of I-94</td>
<td></td>
</tr>
<tr>
<td>Almost every day</td>
<td>28.6</td>
</tr>
<tr>
<td>Almost every week</td>
<td>31.6</td>
</tr>
<tr>
<td>Almost every month</td>
<td>17.7</td>
</tr>
<tr>
<td>Once or twice a year</td>
<td>18.6</td>
</tr>
<tr>
<td>Less than once a year</td>
<td>3.5</td>
</tr>
<tr>
<td>Total</td>
<td>100.0</td>
</tr>
<tr>
<td>Trip purpose</td>
<td></td>
</tr>
<tr>
<td>Social-recreational</td>
<td>34.6</td>
</tr>
<tr>
<td>School</td>
<td>5.7</td>
</tr>
<tr>
<td>Shopping</td>
<td>1.9</td>
</tr>
<tr>
<td>Business</td>
<td>9.5</td>
</tr>
<tr>
<td>To or from work</td>
<td>37.3</td>
</tr>
<tr>
<td>Miscellaneous</td>
<td>11.0</td>
</tr>
<tr>
<td>Total</td>
<td>100.0</td>
</tr>
<tr>
<td>Opinion of motorist aid phones</td>
<td></td>
</tr>
<tr>
<td>Necessary service, should be expanded</td>
<td>69.2</td>
</tr>
<tr>
<td>A convenience, would like to see it expanded</td>
<td>28.2</td>
</tr>
<tr>
<td>A convenience, but not necessary</td>
<td>1.7</td>
</tr>
<tr>
<td>Prefer past methods of obtaining aid, such as raised hood, flare, or handkerchief on door</td>
<td>0.9</td>
</tr>
<tr>
<td>Total</td>
<td>100.0</td>
</tr>
</tbody>
</table>

The seasonal change in the composition of traffic is shown strongly in the high social-recreational percentages in the summer and the high business and work group percentages in the winter.

ANALYSIS OF REPORTS RECEIVED FROM STATE POLICE

Data recorded by the State Police dispatcher on the interview form shown in Figure 21 are given in Table 8. The call distribution rates vary somewhat along the study section; however, it cannot be stated with any confidence on the basis of these variances that the closer spaced phone sites (1 through 34) provide a better service to the motorist than the longer spaced sites (35 through 62). The categories and percentages of aid requested are similar to those reported by most other motorist-aid system studies. Tires and gas are about 21 percent each and mechanical aid, about 36 percent. The 7.3 percent of the calls for accident aid seems rather high for this type of need; information is not presently available concerning details of these accidents.

ANALYSIS OF MAILED QUESTIONNAIRE RESPONSES

An analysis of data received from questionnaires (Fig. 22) mailed to those who used the motorist-aid phone system in 1968 is given in Table 9. Only 73 percent of the phone users were aware of the aid phones before they stopped. The data indicate that those who were in need of aid placed a high value on the motorist-aid system; 69.2 percent considered the system a necessity, and 28.2 percent more thought the system a convenience that should be expanded. Almost all of the users, 97.6 percent, replied that they would use the phones again if the need arose.
Designing the First FLASH Installation

IVOR S. WISEPART, Airborne Instruments Laboratory, A Division of Cutler-Hammer, Farmingdale, Long Island, New York

The feasibility has been demonstrated of a technique that relies on passing motorists to report vehicles needing help. The system is named FLASH, which is an acronym for Flash Lights And Send Help. This paper describes the design and operation of the first installation on Interstate 4 between Lakeland and Orlando, Florida.

THE PROBLEM of quickly detecting, locating, and aiding disabled vehicles on limited-access highways has received considerable emphasis in recent years. In 1962, Airborne Instruments Laboratory (AIL) began an investigation of the extent of the disabled-vehicle problem and a review of the various detection techniques and their economics (1).

COOPERATIVE-MOTORIST FEASIBILITY

In 1966, the Bureau of Public Roads asked AIL to investigate a technique that could be used in detecting and locating vehicles needing help, that was safe and simple, and that could be implemented quickly with minimum equipment cost. The technique was to rely on passing motorists to spot vehicles needing help and to report them at convenient locations along their route. It was desired that these passing motorists remain in their cars without slowing down or deviating from predetermined trip plans. Quick implementation required that no new equipment be installed in the vehicle. The use of familiar instruments, such as lights and horns, also minimized the need for special training of the motorist. Because many miles of rural roads are in desolate areas, it was desired that motorists needing help remain with their vehicles and not abandon them to seek assistance.

AIL conducted experiments on short sections (3 to 6 miles long) of the following routes: Long Island Expressway in New York, I-70 in Kansas, I-80 in Nebraska, I-15 in California, and Richmond-Petersburg Turnpike in Virginia. These roads were selected to cover a cross section of highway types and user characteristics. Experimental signs were installed requesting motorists to flash their lights (or sound their horns) 3 times if they saw vehicles needing help. A vehicle and driver were staged along each test section, simulating various scenes of disablement. Observers at the reporting location recorded the responses of the passing motorists. The data recorded indicated that passing motorists could be relied on to report motorists needing help and that the cooperative-motorist concept was indeed feasible (2).

THE FIRST OPERATIONAL INSTALLATION

After the operational feasibility of the cooperative motorist technique had been proved, the next logical step was to design and install the equipment and evaluate a fully operational system.

A 50-mile segment of Interstate 4 between Lakeland and Orlando, Florida, was selected because it satisfied the criteria for an initial installation. Factors included in the selection were number of interchanges, interchange spacing, traffic volume, and proximity to major cities at both ends of the test section. The varied availability of motorist services at interchanges along I-4 was a characteristic typical of rural Interstate mileage.

Paper sponsored by Committee on Communications and presented at the 49th Annual Meeting.
Local support was a basic requirement for achieving a successful first installation. The Florida Department of Transportation recognized the need for a disabled-vehicle location system and made important contributions to the system design and installation. The Florida Highway Patrol is closing the system loop by operating the terminal equipment and responding to motorists' needs for help. This combined effort will be the means for achieving the goals of public understanding, cooperation, and confidence in the FLASH System.

Figure 1. Content and typical locations of roadside signs.
INFORMING MOTORISTS

The successful operation of the FLASH System depends on the participation of the motoring public. Maximum participation of motorists can be achieved only through an extensive public education campaign and through widespread and uniform adoption of the FLASH System. Because this is not practicable for this first demonstration installation, conventional methods for soliciting motorists' participation have been adopted.

Motorists traveling along I-4 are informed how to report by a sequence of roadside signs (Fig. 1). Considerable attention was given to the design of these signs for maximum effectiveness. They are designed fully in accordance with pertinent Interstate signing specifications and current safety standards for placement and construction. The characters are reflectorized, and blue reflective background is used as indicative of motorist services. The signs have 30-ft offsets and breakaway support structures.

Sign 1 is located so that motorists entering I-4 will be quickly informed what to report (Vehicles Needing Help) and where to report (At FLASH Sign). As motorists approach the subsequent interchange, sign 2 relates how they should report (Flash Brights 3 Times) and repeats where and why. Sign 3, the FLASH sign, is located about one-quarter mile beyond sign 2 to allow cooperative motorists sufficient time to prepare to flash their bright lights.

The FLASH sign is intentionally designed to attract attention by its nonuniform shape and color. The sign has 15-in. reflective blue letters on a reflective white elliptical background. The elliptical shape has major and minor diameters of 100 in. and 40 in. respectively. It is intended that widespread application and motorist familiarization with the operation of the FLASH System will require only the presence of FLASH signs to designate the reporting location. Thus, future installations will use instructional signs at infrequent intervals as a reminder or be eliminated entirely.

Signs have been placed selectively (Fig. 2) to permit testing of the motorist's comprehension and retention of the sign instructions. The 50-mile section has 20 FLASH reporting stations—10 in each direction for an average 5-mile spacing. Each reporting station is designated by the presence of a FLASH sign. Instructional signs 1 and 2 are frequent at the beginning of the section. Toward the end of the section, signs 1 and 2 occur only after major interchanges that generate new traffic. Sign 4 (End FLASH Area) is located at each end of the instrumented section.

FLASH EQUIPMENT

In the design of the equipment that transmits motorists' flashes to Highway Patrol Troop Headquarters, maximum consideration was given to reliability, maintainability, compactness, and simplicity of operation. For example, wherever possible, throwaway plug-in modules and integrated-circuit components are used. The basic

![Figure 2. Test section and sign and detector layout.](image-url)
equipment required consists of roadside detectors, roadside computers, a central processor, and a monitor console.

**Detectors**

Associated with each FLASH station are a detector and roadside computer. Because motorists must be within the effective detection area (Fig. 3) for their flashes to be counted, it is necessary to determine the optimum detector location that would account for variations in motorists' initial flashing positions and their rates of flashing. Preliminary tests conducted on I-4 during August 1968 indicated that this optimum location was 350 ft beyond the FLASH sign.

The detector installation (Fig. 4) is designed and located to be as inconspicuous as possible and to minimize vandalism. Thus, the detector is placed so as to exactly substitute for a delineator and supports the delineator reflector. Because the detector is a line-of-sight device, each location must be carefully selected with consideration given to both horizontal and vertical roadway alignment.

Physically, the detector is a 4-in.-diameter plastic cylinder about 48 in. high. If hit by a vehicle, a notch cut around its perimeter will shear the detector off at the base. At the same time, a connector supplying power to the detector will also separate, and a signal will be transmitted to the monitoring console to inform the observer of the event. If the impact has not damaged the detector tube or its electronic operation, it can be reused by covering the separated pieces with a short plastic sleeve.

Photosensitive cells within the detectors sense motorists' flashes during the day or night and send signals through underground cabling to roadside computers.

**Roadside Computers**

Roadside computers, located at the edge of the right-of-way near the detectors, discriminate between valid flashes from cooperative motorists and spurious alarms caused by random reflections, lightning flashes, and the like. If a detector receives 3 flashes within a 5-sec interval, its roadside computer will transmit a coded tone signal through telephone lines to a central monitoring station located at the Florida Highway Patrol Troop Head-
quarters in Lakeland. Power requirements for each roadside computer and detector combination are less than 40 watts.

Central Processor

The equipment in the Lakeland Headquarters consists of a central processing unit and a monitor console. When the central processor receives a report signal from the roadside computer, an electronic timer is started. False-alarm indications are minimized by the requirement that valid reports be received from more than 1 vehicle before a disable-vehicle alarm is generated. False alarms may be caused by pranksters, curious motorists, and well-meaning motorists who misunderstand instructions.

The number of cooperative vehicles required to generate a disabled-vehicle alarm and the timer interval vary depending on the highway and user characteristics. For example, on high-volume road sections, 4 or 5 vehicles must flash within 3 min to generate a disabled-vehicle alarm; on low-volume roads, 2 or 3 vehicles must flash within 5 min. Each FLASH station can have its own setting; a HI/LO TRAFFIC switch on the monitor console permits the setting to be changed easily to account for volume variations caused by day and night cycles, inclement weather restrictions, and the like.

Monitor Console

The monitor console (Fig. 5) in the Lakeland Troop Headquarters is within arm’s reach of the radio operator. On the console face panel, 2 horizontal light strips represent the eastbound and westbound directions of I-4. Vertical plastic strips represent the interchange crossroads. A pair of illuminated pushbutton switches is associated with each section of I-4 having a FLASH station.

When the road is clear of disabled vehicles, the I-4 light strips are illuminated green. When a disabled-vehicle alarm is received, a momentary tone sounds to alert the radio operator, the section of I-4 associated with the alarm signal turns from green to red, and the upper pushbutton adjacent to the highway section illuminates red with the word DISPATCH. After the radio operator dispatches a vehicle to investigate the red section, he presses the DISPATCH pushbutton to extinguish it and illuminate in amber the lower pushbutton with the word SERVICE. When the dispatched vehicle driver reaches the disabled motorist and ascertains the trouble, he informs the radio operator who then presses the SERVICE pushbutton. This extinguishes the SERVICE pushbutton and the roadway section returns to its normal green color.
SYSTEM TEST

The faithful operation of each detector link can be ensured by monitoring the progress of a test vehicle that flashes 3 times as it passes each detector. When the TEST switch on the console is in the TEST position and a valid report is received, a small indicator light adjacent to each pushbutton switch flashes at a fixed rate. This simple test checks out the detector alignment and optics, the underground roadside link, the telephone communications link between the detector and the monitor station, and the monitor station electronics. If a detector is knocked down, the associated indicator light illuminates continuously, informing the observer of the exact detector involved.

EVALUATION EQUIPMENT

During the first 12 months of system operation, AIL will collect data on system effectiveness and operation. Data will be recorded on punched paper tape. The number of vehicles flashing during the preceding 1-min interval will be recorded every minute of the day for each FLASH station together with radio-operator console manipulations.

FLASH SYSTEM EVALUATION

During the 12-month evaluation period, which began on November 14, 1969, we will determine the extent to which the FLASH System is improving service to stranded motorists. This will be accomplished through analysis of the punched paper tapes and also of Highway Patrol reports, staged experiments, and motorist questionnaires and interviews. The system will be refined during the evaluation period as operating experience is developed. As operators of the system, the Florida Department of Transportation and Highway Patrol will be consulted for their suggestions. The cooperation of the mass media will be solicited to assist in indoctrinating the public to the FLASH System through a carefully planned and coordinated publicity effort.

CONCLUSIONS

The greatest advantage of the FLASH System is that all road vehicles are presently equipped to participate and that minimum learning is required of the driver. This system is a tool that will enable the highway patrol to use its equipment more efficiently and thereby provide faster service for assisting the disabled motorist. This indirectly leads to increased highway safety.

This system is suited for use on highways in rural areas, including those with tourist traffic. The tourist, who is unfamiliar with the location of service facilities in the area, simply stays with the disabled vehicle and waits for official assistance to arrive.

Because this is a prototype system, the major effort has been concentrated on the development of reliable detection and monitoring. Future roadside detector stations could have a self-contained power source and communication link. This would facilitate the installation of detector stations in areas where power and telephone service are not available.

In the final analysis, the public will judge the acceptability of the system through its expression of confidence and satisfaction with the improved service provided.

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REFERENCES