Capacities and Characteristics of Ramp-Freeway Connections

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This report presents some of the initial findings of the nationwide Freeway Ramp Capacity Study, sponsored jointly by the Highway Research Board and the U.S. Bureau of Public Roads, for which data were gathered in 1960 and 1961. Additional data recently collected by the Bureau of Public Roads have been incorporated into some of the equations. The capacities associated with ramp-freeway connections are described. These capacities are as follows: for entrance ramps—(1) capacity of the entrance point from the arterial or freeway supplying traffic to the ramp, (2) capacity of the "ramp proper" and (3) capacity of the merging operation at the freeway terminal of the ramp; for exit ramps—(4) capacity of the diverging movement from the freeway to the ramp, (5) capacity of the "ramp proper" and (6) capacity of the ramp terminal connection to the street system.

Any one of these capacities can be the limiting capacity of a ramp. In this study, the emphasis is on the capacity of the merging operation at the freeway terminal of the ramp. In addition, there is some discussion of the capacity of the diverging movement from the freeway to the ramp. Detailed analysis of diverging capacity is under way.

The merging and diverging capacities to and from the freeway not only reflect ramp performance, but also have an important effect on the capacity of the freeway lane. For an entrance ramp, merging capacity is a measure of the ability of the ramp vehicles to make the transition to freeway operation. For an exit ramp, the diverging capacity is a measure of the ability of the freeway vehicles to disengage from the freeway flow and follow their intended path along the ramp.

Before merging capacities can be computed, the freeway lane volume distribution must be known so that lane 1 (i.e., the shoulder lane or righthand lane) volume can be estimated for the given freeway volume. These percentage distributions for four-, six-, and eight-lane freeways are depicted by graphs. As an alternative method in estimating lane 1 volume, equations are presented for use when certain upstream and downstream adjacent ramp conditions are known. These equations make possible an increase in the accuracy of the lane 1 volume calculation. Several of the equations are presented in nomograph form.

Curves are presented showing the free-flowing capacity of various interchange on-ramp connections for different proportions of ramp and lane 1 volumes. In the one group of curves, volume in lane 1 is the independent variable; in the other, volume on the ramp is the independent variable.

Two formulas determined by regression analysis are presented for use in determining free-flow merge capacity at one-lane on-ramps. The formula variables are discussed as to their relative importance and their use is outlined in a sample problem.

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Two-lane ramp operations (both on and off types) comprise 42 of the 219 separate studies submitted. These ramps varied widely in both geometrics and traffic characteristics. There were insufficient numbers in any one category to permit derivation of capacity formulas. Several ramp lane distribution curves are shown for some individual two-lane ramp studies. Three of the most interesting two-lane on-ramps are discussed and volumes are quoted. Some general conclusions are drawn from the two-lane ramps submitted.

Finally, several diamond ramps on the Edsel Ford Expressway are offered as representing the type of efficient operation which should be attainable under desirable conditions.

•THE INITIAL concept of a freeway ramp capacity study was developed jointly in August 1958, by O. K. Normann, chairman of the Committee on Highway Capacity and by its Subcommittee on Ramps, under the chairmanship of Leo G. Wilkie. The preliminary study forms were prepared by Mr. Wilkie and presented for consideration at the January 1959 meeting of the committee. The need for a comprehensive picture of ramp-freeway interaction was stressed at that time.

Recognizing this need, the Bureau of Public Roads assumed responsibility for the study. The final layout of field forms and instructions was completed in June 1960. The field phase of the study, carried out by the States and municipal organizations, began shortly thereafter. The Highway Research Board and the Bureau of Public Roads collaborated in bringing the project to the attention of State highway officials and municipal organizations. The data from the first field studies were received in September 1960; as of October 1962, data were received for 219 studies conducted at 195 ramp-freeway connections.

COLLECTION OF DATA

Participating Agencies

The following State highway departments and municipal agencies collected data for this study: California, Colorado, Connecticut, Florida, Georgia, Illinois, Indiana, Kansas, Maryland, Michigan, Minnesota, Missouri, New Jersey, New York, Oregon, Pennsylvania, Rhode Island, Texas, Virginia, District of Columbia, Port of New York Authority, and Cook County (Ill.) Highway Department. Junior engineer trainees of the Bureau of Public Roads studied several locations in Virginia, and eleven locations in Detroit were studied by the author in cooperation with personnel from the Michigan State Highway Department.

Field Procedure

Studies were conducted at both on-ramp and off ramp junctions with freeway, parkway, and expressway facilities. Traffic counts were made by continuous 5-min increments at the nose of the ramp, each observer usually counting one lane of traffic but never more than two lanes. Counts usually began about 30 to 60 min before the peak hour started and continued beyond the peak hour by about the same interval. At on-ramps counts were made at a point just before the nose of the ramp where physical separation still existed between the two flows. At off-ramps, counts were made just downstream from the nose, after physical separation had been established. Figure 1 shows counting locations at both on-ramp and off-ramp locations.

There was considerable variation in speed-recording procedures. Radar speed meters were commonly used, but a number of States used stopwatch time measurements over a measured distance. Several studies were conducted with speeds estimated by observers. Camera and traffic analyzer methods were also used to some extent.

Vehicles were classified as passenger cars or commercial vehicles, the latter including any vehicle with more than four tires.

At each study location an experienced observer kept notes describing within each 5-min counting increment the operation at the study area. Conditions upstream or downstream were also noted, especially when they affected the main study location. The observer's duty was to report any apparent reasons for congested operation although he was cautioned not to speculate. The observer's remarks, along with the recorded speeds, were used as guideposts in identifying the 15-min free-flow periods.

Adjacent ramps, both upstream and downstream, were usually counted simultaneously with the main study location. Remarks on the traffic operation were also made at these adjacent ramps, although freeway lane counts were not taken. At some locations counts were also made at the ramp terminal connection with the local street system. These counts served as a check on the main study area ramp counts and indicated the ability of the discharge point to handle the ramp traffic.

It was decided that continuous counts over 2- or 3-hr periods would be more accurate than short counts interspersed with rest and recording periods which would require interpolation of the data. Because high-volume periods of at least 15 min of free flow were desired, continuous counts and remarks were needed to accurately delineate these periods.

GLOSSARY

The terms used in this report are defined here for ready reference, as follows:

Angle of convergence: The interior angle made between the right edge of lane 1 and the left edge of the ramp at right-hand on-ramps. Where the ramp and/or freeway is on a curve at the nose, a 100-ft chord is drawn back from the nose to its intersection with the inside edge of the ramp and/or freeway lane. The interior angle formed by the chords or by the chord and the tangent edge of the ramp or freeway is then measured as the angle of convergence. The use of the 100-ft chord is an arbitrary choice as an estimate of the average driver's path. The angle of convergence can also be computed if the design radius of the ramp curve and the ramp width at the nose are known. Assuming the design radius given is that for the inner edge of the ramp, the formula would be:

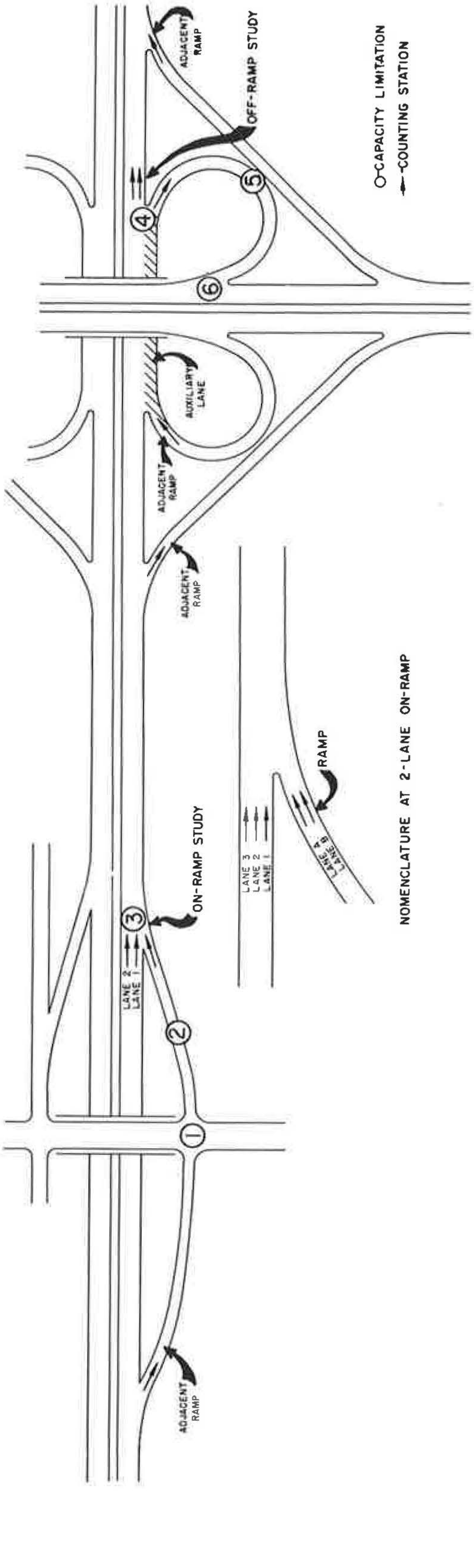


Figure 1. Capacity limitations and counting locations for on-ramp studies.

$$\sin(\text{angle of convergence}) = \frac{50}{\text{Ramp radius (ft)} + \text{Ramp width (ft)}}$$

An alternative would be to consult a table of radii in a surveying textbook, using the chord definition of the radius in getting the degree of curve. The denominator of the above formula would be looked up and the angle of convergence would be ½ D (degree of curve).

Angle of divergence: The interior angle made between the right edge of lane 1 and the left edge of the off-ramp. If either is curved, a 100-ft chord should be employed under the same reasoning as applied to "angle of convergence".

Free-flow merge: Condition where freeway traffic is moving in a uniform manner somewhere in the 35- to 60-mph range. Large fluctuations in speeds are few and traffic is experiencing no conflicts severe enough to cause intermittent braking or congestion. Ramp traffic flow, though possibly slower in speed than the freeway, is continuous without backup on the ramp. The merge of the two streams is normally smooth within the usual adjustments in speed necessary for this maneuver. No specific overall speed should be associated with "free flow," as the design and type of interchange will have an important effect on the speed at any one location. The free-flow periods chosen for this study are of 15-min duration and these volumes are expanded to one hour by multiplying by four (15-min f.f. exp.) The operation during a free-flow period is assumed to be capable of continuance, barring increased demand, backup from downstream, or vehicular accidents. Yet volumes will be in the practical to possible capacity range so that increased demand could cause a breakdown in the operation.

Lane 1: The right-hand lane of the freeway.

Lane 2: The second lane from the right-hand edge of the freeway.

Lane 3: The third lane from the right-hand edge of the freeway.

Median lane: The lane adjacent to the median. In the case of a 6-lane freeway, the median lane would be lane 3.

Peak merge hour: The hour of the highest merge (lane 1 + ramp).

Percent commercial vehicles in merge (% c.v. in merge): The number of commercial vehicles in the merge divided by the total number of vehicles in the merge:

$$\%\ \text{c.v. in merge} = \frac{\text{c.v. (Ramp + Lane 1)}}{\text{Merge volume (Ramp + Lane 1)}} \times 100$$

Percent freeway utilization (% fwy. util.): A measure of the freeway use immediately upstream from the on-ramp nose. It is the hourly freeway volume or 15-min f.f. volume expanded to 1 hour divided by the number of lanes multiplied by 2,000 vph/lane possible capacity per lane:

$$\%\ \text{Fwy. util.} = \frac{\text{Fwy. volume (vph)}}{\text{No. lanes} \times 2{,}000\ \text{vph/lane}} \times 100$$

Ramp lane A: The ramp lane closest to the freeway in the case of a two-lane ramp (see Fig. 1).

Ramp lane B: The ramp lane farthest from the freeway in the case of two-lane ramps (see Fig. 1).

Ramp/merge ratio: A measure of the merge components consisting of

$$\frac{\text{Ramp volume}}{\text{Merge volume (Ramp + Lane 1)}} \times 100$$

Rate of flow or hourly rate: The volume for a short period of time, such as 5 or 15 min, expanded to a vehicles-per-hour figure by the factor

$$\text{Short period volume} \times \frac{60}{\text{Short period (minutes)}}$$

ON-RAMPS

These data were collected under a nationwide "freeway ramp capacity study." In retrospect, this appelation was misleading because it gave the impression that the primary reason for the study was to determine a specific capacity of the "ramp proper." Although this was one objective of the study (if, in fact, such a value can be established), it was a relatively minor objective compared to the need for capacity figures at the merging and diverging ends of the ramp. It was this need which was the primary motivation for this project.

At on-ramps there are the following possible capacity limitation locations (circled numbers, Fig. 1):

1. The entrance point from the arterial or freeway supplying traffic to the ramp.
2. The ramp proper.
3. The merging operation at the freeway terminal of the ramp.

The first of these is outside the scope of this study if the traffic is supplied via a traffic signal system or an ordinary street network. If the ramp traffic is supplied by another freeway or expressway, the "diverging" from that facility is within the scope of this study.

The capacity of the ramp proper is still thought of by some engineers as the limitation of a ramp's ability to carry traffic. In a sense they are right because it is the ultimate capacity limitation, and in a few cases, where there are no limitations to a free flow at either end, this does become the limiting capacity of a ramp. However, conditions at the ramp terminals usually preclude any possibility of obtaining this capacity, making it nearly meaningless from an operational or design standpoint. Unless an additional through-lane is provided beginning at the entrance terminal, an on-ramp is seldom completely loaded with traffic; in most instances where this does occur, it is because the ramp vehicles cannot merge onto the freeway. One of the ramps in this study which did reach the capacity of the "ramp proper" was the cloverleaf inner loop connection from the Long Island Expressway westbound to the Brooklyn-Queens Expressway southbound. This ramp carried 1,918 vehicles in the peak hour, because the capacity restraints at its terminals were removed. At the exit from the Long Island Expressway to the ramp, police directed the outside expressway lane into the ramp. At the other end, the two ramp lanes were necked down to one lane by paint striping before the entrance to the Brooklyn-Queens Expressway. Here, lane 1 of the expressway was coned off, permitting free access for the continuous stream of ramp vehicles. It is reasonable to assume that the volume of traffic handled by this ramp in the peak hour would be considerably less if these unusual steps had not been taken.

The capacity of the merging operation at the freeway terminal of the ramp is most important from the standpoint of the entire freeway system. It is this merging capacity which is examined most thoroughly in this study. Along with weaving, it is one of the most troublesome problems encountered in freeway operation. A faulty merging operation at the freeway terminal of the ramp not only disrupts smooth operation along the freeway but can also cause a backup along the ramp, sometimes extending far enough to block the cross street, frontage road, or lane 1 of an interchanging freeway.

The emphasis in this study is on free-flowing capacity. Perhaps it best corresponds to the 1,500-vph/lane concept of urban "practical capacity." However, the volumes in free-flowing capacity as used in this report ranged up to the values commonly associated with "possible capacity," and in a few instances to less than practical capacity.

A "free flow" is assumed to exist when the following conditions are present: (1) The freeway traffic is moving in a uniform manner somewhere in the 35- to 60-mph speed range; (2) Large fluctuations in speeds are few and traffic is experiencing no conflicts severe enough to cause intermittent braking or wave action; (3) Ramp traffic flow, although possibly slower in speed and more erratic than that on the freeway, is continuous within the ramp demand range without backup on the ramp; and (4) The merging of the two traffic streams is normally smooth within the usual range of adjustments in speed necessary for this maneuver.

No specific overall speed should be associated with free flow, inasmuch as the design and type of interchange has an important effect on the speed range that can be associated with free flow at any one location.

The free-flow periods chosen for this study are of 15-min duration, consisting of three consecutive 5-min counting periods. These 15-min volumes are expanded to 1-hr volume rates by multiplying by four. Hence, the phrase "15-min free-flow expanded period" used throughout this report. The operation during a free-flow period is assumed to be capable of continuance, barring increased demand, backup from downstream, or vehicular accidents. Yet volumes will be in the practical to possible capacity range (urban definition), so that increased demand could cause a breakdown in the operation.

Although there is increasing awareness among those working in the highway capacity field of the need for a peak short-period factor to be applied to design-hour volumes, such a factor was not incorporated in this report for two reasons. First, no nationally applicable procedures for its application have yet been developed. Second, some of the 15-min periods used in this analysis were isolated periods falling outside the actual peak hour, which would have complicated the development of a factor.

Nevertheless, the effect of such a factor, if developed, should be noted. It would allow for the short-term high-volume peak found within the peak hour. The facility would thus be able to continue efficient operation throughout the short-period peak. The short-term peaks as taken from the data reported in this study were higher in the smaller cities (Fig. 2). Free-flow and non-free-flow curves are based on metropolitan area populations. For example, the free-flow curve shows that the 5-min peak volume can be expected to approximate 10.65 percent of the peak-hour volume for a metropolitan area of 250,000, but would only be 9.63 percent for 5,000,000. The standard error on both curves is 8.0 percent above and 7.4 percent below the curve.

If a 6-lane freeway within a metropolitan area of 3,200,000 population carries 5,400 vehicles (1-way) in a peak hour, the 5-min peak volume would be 528 vehicles (9.78% × 5,400) using Figure 2. The hourly rate for the 5 minutes would be 6,336 vehicles (12 × 528). Although it is sometimes possible to sustain this 2,112 vph/lane average for a short period of time on a well-designed freeway, such short-term loads can easily precipitate a stop-and-go type of operation. Application of the factor would reduce the likelihood of this occurring, but would also reduce the design-hour volume. The present AASHO design-hour volume of 1,500 vph/lane for urban freeways was chosen to allow for this short-term peaking.

Maintenance of free-flow operation is not always possible at any selected study point even though the basic ingredients for high-level operation are present. Backups from points downstream can cause congestion for several miles upstream as queueing develops. The ability of the freeway to carry a large volume of traffic past a point is not necessarily hampered by the stop-and-go type operation seen on some freeways during rush hours. Volumes of 1,900 to 2,200 vph/lane are possible, but speeds will be reduced and travel time increased. Higher volumes are often obtained during these congested periods simply because there is a continual steady demand on the facility (Appendix C). There is no need to enumerate the disadvantages of this congested kind of operation which are reflected primarily in traffic delays. The same facility operating with free flow at volumes between practical and possible capacity will still have numerous large relatively open stretches where freeway utilization is low. These open stretches are important, in that they

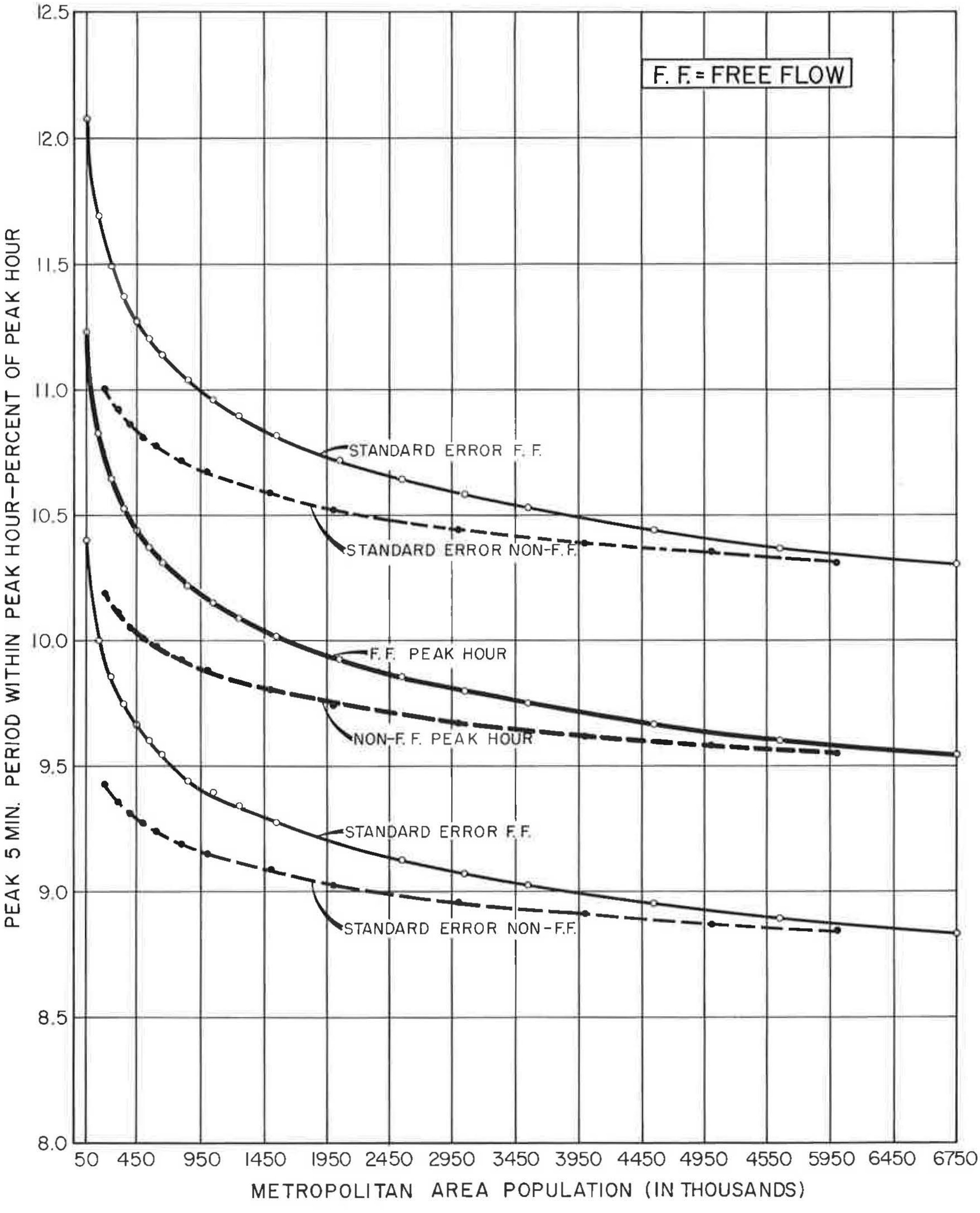


Figure 2. Peaking trends related to population.

allow short periods for recovery of uniform speeds after a momentary surge of ramp traffic has impeded freeway lane 1 speeds. This is especially so at diamond ramps, where platoons of 15 to 20 vehicles are often released by the traffic signal controlling the entrance of traffic from the local street system.

Some of the well-known traffic flow concepts, such as the typical speed/volume relationship, do not always hold true when maneuvering is under way in interchange areas. It is common sense to expect a more erratic operation within complicated interchanges as compared to a simple diamond interchange. City size and driver experience are important in interchange operation: an interchange carrying predominately commuter traffic should have smoother operating characteristics than an interchange geared more toward tourist or interstate traffic.

Poor usage of available speed-change lanes was observed in several studies conducted at ramps serving recreational areas. An example of this was in Michigan at the Kent Lake Road southbound on-ramp to Interstate 96 eastbound. This ramp approximately 20 mi west of Detroit, was studied on a Sunday afternoon when it carries its heaviest volume—Detroiters homeward bound from the Kent Lake recreational area. Many of the 596 peak-hour ramp drivers seemed unaware of the 1,000-ft acceleration lane available for their use, as they either cut directly into lane 1 or stopped at the nose until a suitable gap in traffic appeared. Although the peak-hour freeway volume was only 1,646 vehicles for two lanes, ramp drivers had a difficult time because an entire platoon of vehicles would often be held up by a lead driver who stopped. Granted that the high speed (50- to 60-mph range) of the Interstate 96 vehicles was an inhibiting factor, this facility should still have operated satisfactorily considering the volumes and geometrics. The poor operation seems more a result of driver unfamiliarity with this particular interchange than of overall driver inexperience with interchange driving, because many of these drivers no doubt have had considerable experience on the Detroit expressway system.

Although different lane design volume levels have been established for urban, suburban, and rural locations, there is no easily applied factor to account for driver unfamiliarity. Making the drivers' optimum path readily apparent is always important, but it appears to have even more importance at locations similar to the Kent Lake interchange. Aside from capacity considerations, recognition should be given to the relatively unsafe operation which results when stopped or low-speed ramp vehicles attempt to merge into high-speed through traffic.

Freeway Volume Distribution by Lanes at On-Ramps

The freeway volume distributions by lanes are given in Figures 3, 4, 5, 6, and 7. These freeway volume percentages are as taken just upstream from the ramp nose before merge has taken place. Of course, the volume in lane 1 has a marked effect on the merging operation and the greatest possible accuracy is needed in determining lane 1 volumes at the ramp nose.

For 4-lane freeways, the freeway volume distributions are presented in two groups (Fig. 3)--those at cloverleaf inner loop on-ramps and those at all other types of on-ramps. The reason for this grouping is the difference in operation at cloverleaf interchanges caused by traffic weaving between the adjacent inner loops. In comparison with other types of ramps, the cloverleaf inner loop ramp curves show a heavier use of lane 1 up to freeway volumes of 2,400 vph, despite the loss of lane 1 vehicles at the upstream adjacent outer connection off-ramp. Much of lane 1 traffic is destined for the downstream inner loop off-ramp only 400 to 700 ft away. At freeway volumes above 2,400 vph the comparison shows a heavier use of lane 2 at cloverleaf locations, possibly because drivers wish to avoid the more severe merging and weaving conflicts present at high-volume cloverleaf interchanges.

Three sets of curves are shown for 6-lane freeways at on-ramp locations. Figure 4 contains data for freeways at diamond on-ramp locations only. Figure 5 is derived from 6-lane freeway volume distributions at all types of on-ramps, including diamond ramps but excluding cloverleaf inner loops. Figure 5 also gives the volume distributions where an auxiliary lane is present between the on-ramp and the adjacent downstream off-ramp. Lane 1, where paralleled by an adjacent auxiliary lane starting at

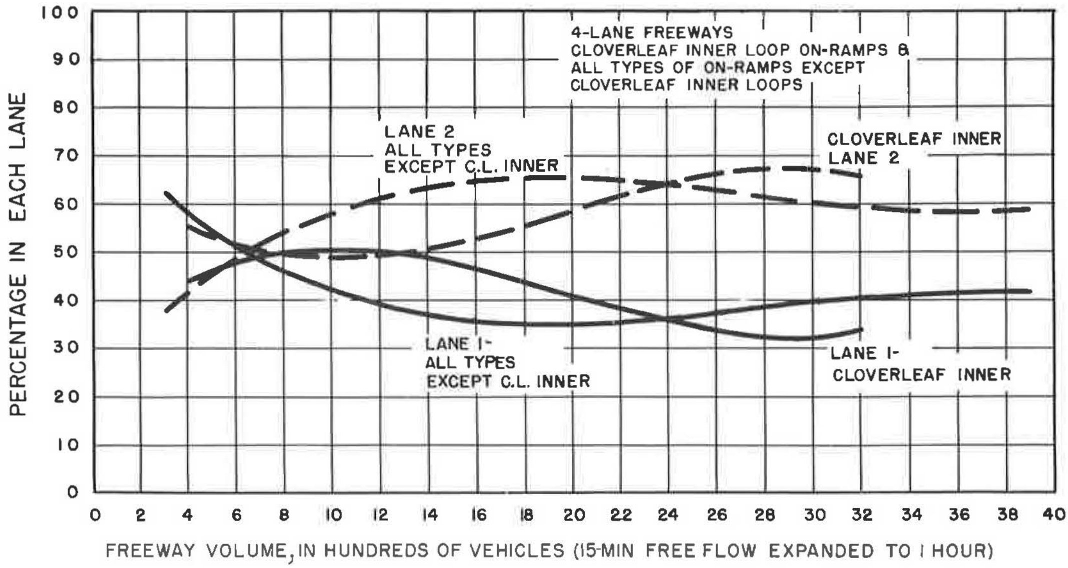


Figure 3. Volume distribution on 4-lane freeways upstream from cloverleaf inner loop and from all other types of on-ramps.

the on-ramp junction, carries approximately eight percentage points more of the freeway traffic as counted at the ramp nose than where there is no auxiliary lane. Investigation disclosed this extra amount approximated the volume exiting at the adjacent downstream ramp at the end of the auxiliary lane. It should be emphasized, however, that the auxiliary lanes contained in the data were of the shorter lengths (none exceeding 1,000 ft) and longer ones might result in a different freeway lane volume distribution. Figure 6 gives lane volume distributions upstream from cloverleaf inner loop on-ramps having auxiliary lane connection with the inner loop off-ramp.

The curves for 8-lane freeways (Fig. 7) should be compared with those for 6-lane freeways. At a given freeway volume the 8-lane freeway will carry less in lane 1 up-

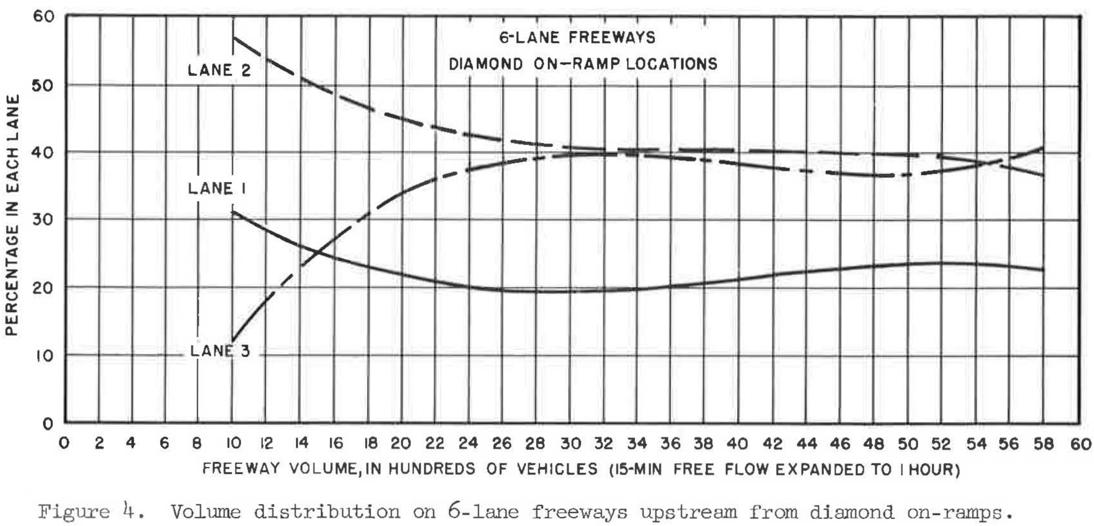


Figure 4. Volume distribution on 6-lane freeways upstream from diamond on-ramps.

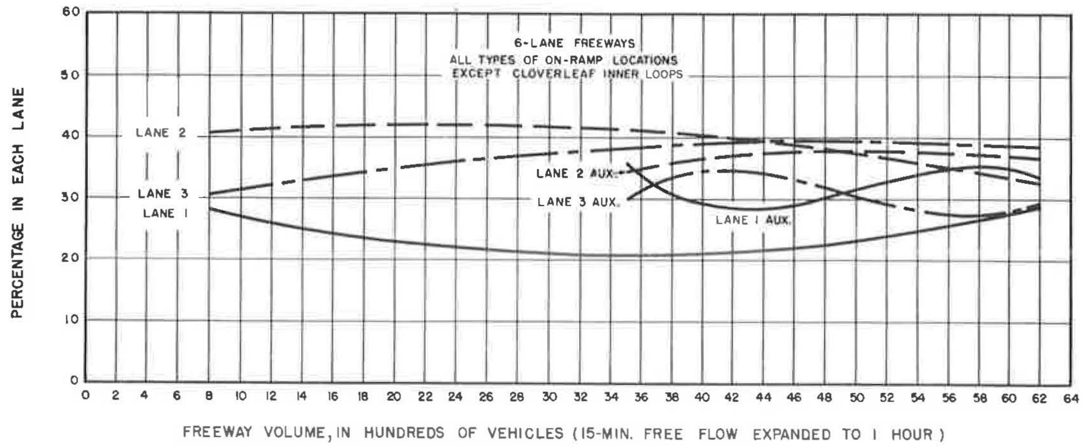


Figure 5. Volume distribution on 6-lane freeways upstream from all types of on-ramps (with and without auxiliary lane at on-ramp entrance) except cloverleaf inner loops.

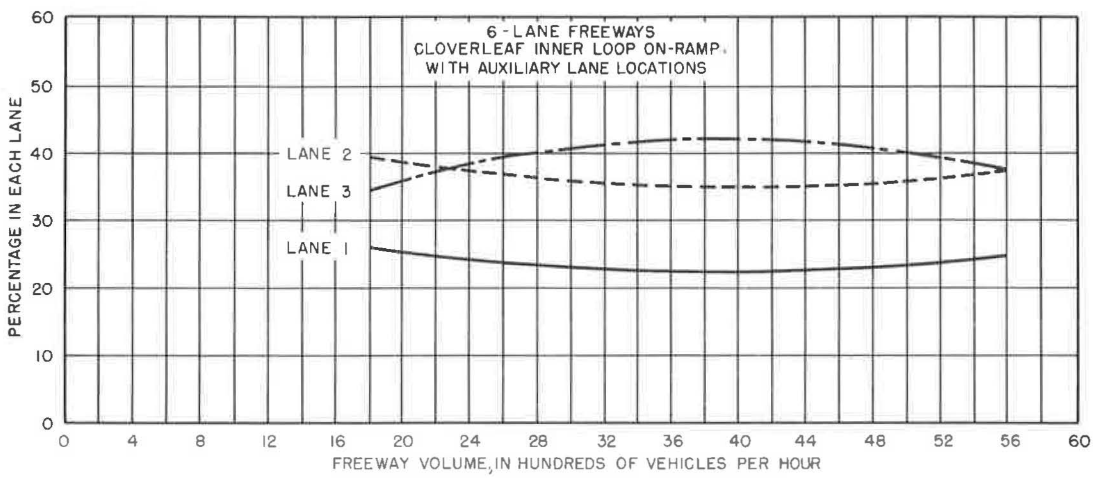


Figure 6. Volume distribution on 6-lane freeways upstream from cloverleaf inner loop on-ramps with auxiliary lane.

stream from the on-ramp because there is an additional high-speed inner lane carrying traffic. Thus, in effect, a higher entrance ramp volume can be accommodated. However, at a given freeway lane volume average, such as 1,500 vph/lane (i.e., 6,000 vph for the 4 lanes of an 8-lane freeway and 4,500 vph for the 3 lanes of a 6-lane freeway), there will be little difference in the lane 1 volume upstream from the entrance ramp and thus little difference in the ramp volume which can merge onto the freeway.

The freeway lane volume distribution at on-ramps varies more at the lower freeway volumes. As an example, data taken at 12 study locations on 6-lane freeways at diamond on-ramps (Fig. 4) in Atlanta, Buffalo, Detroit, and New York City showed the largest residuals in the least squares fit of the lane volume percentage curves at freeway volumes below 2,500 vph. The curves are calculated using data from 41 different 15-min free flow expanded periods in the 1,120- to 5,920-vph freeway volume range. The calculated percentage for several of the low-volume periods differed by

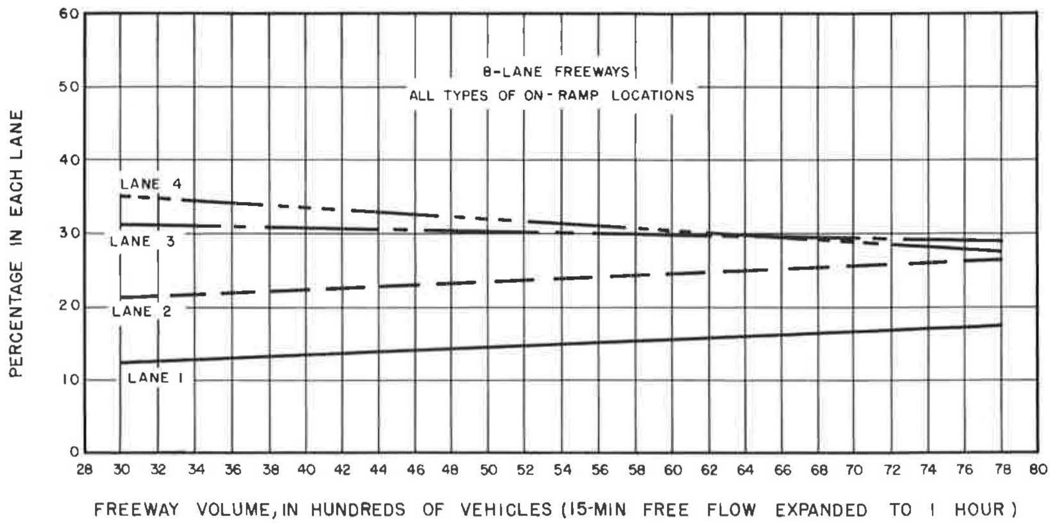


Figure 7. Volume distribution on 8-lane freeways upstream from on-ramps.

as much as 7 to 13 percentage points from the actual percentage. Overall, the standard error of estimate was 2.7 percentage points for lane 1, 3.1 for lane 2, and 3.6 for lane 3.

The behavior of drivers at low freeway volumes cannot be predicted as accurately as at higher volumes because of the great freedom of movement possible. External factors will often influence the choice of a lane for traveling on the freeway. This choice can be exercised at lower volume levels. However, once volumes build up, the individual driver becomes more restricted by his fellow drivers, who are now in closer proximity. The driver's choice of lanes thus becomes more influenced by headways, speeds, and adjacent lane volumes. Once these factors begin to have a pronounced effect on drivers' decisions, there is apt to be less variation in lane percentages between facilities at given volume levels.

The question sometimes arises as to how much effect an on-ramp has on lane 1 traffic. Of course, the lane volume distribution curves do reflect an effect, but what motivates a driver to drive in lane 1 is a question which may never be fully answered. High-volume ramps carrying more than 1,000 vph exert considerable pressure on lane 1 vehicles. Even at more usual ramp volumes, if freeway volumes are light upstream from the ramp there is a tendency for lane 1 vehicles to move over into lane 2 to avoid conflict with the ramp vehicles. This is especially so at low-speed ramp connections, which inhibit through traffic speeds in lane 1. At higher freeway volumes and more usual ramp conditions this tendency is much less pronounced. Commuters' driving habits are fairly well fixed and any tendency to avoid ramp traffic is usually masked because the maneuver to lane 2 or lane 3 may take place well upstream from the ramp. There appears to be a certain amount of local variation in whether drivers move over to avoid ramp vehicles. The degree of conflict the ramp vehicles cause, plus the ease of making a lane change, exert considerable influence on the driver's choice.

Several checks were made on the 6-lane divided Edsel Ford Expressway in Detroit to determine how many cars were moving over within the vicinity of the ramp. In one study, out of 1,003 vph in lane 1, 32 vehicles moved over within the 225-ft stretch upstream from the ramp nose, and 28 others moved over while adjacent to the 575-ft acceleration lane. These 60 vehicles amount to only 6 percent of the lane 1 vehicles moving to lane 2 over the total distance of 800 ft. The ramp volume was 790 vph and the freeway volume 4,372 vph at this location. At another on-ramp in Detroit, 3 percent of 1,426 lane 1 vehicles moved over in the stretch from 100 ft upstream to 300 ft downstream from the ramp nose. The ramp volume was 842 vph and the freeway volume 5,379 vph. It is improbable that all the lane 1 vehicles shifting did so because of the presence of the ramp. Aside from those vehicles which do move over, many lane 1 drivers reflect ramp pressure by edging over near the left edge of their lane at the on-ramp junction. This maneuver gives the ramp vehicle more room laterally in which to jockey while merging. After merging, the majority of the ramp drivers prefer to move over into adjacent lanes as opportunity permits. Studies made recently at four varying freeway sections in Detroit disclosed that approximately 55 percent of the ramp vehicles had moved out of lane 1 within 1-mi downstream from their point of entry onto the freeway. Average freeway lane volumes were in the 1,200- to 2,100-vph range during these studies.

The freeway lane volume distribution curves in this report are least squares fits made without taking into account the variation in ramp volumes. As such, they fairly well represent an average condition. An alternative method for calculating lane 1 volume is given later in this report and in Appendix B. This method takes into account not only the freeway and ramp volumes, but also distances to and volumes of adjacent ramps. Unfortunately, only enough data were available to derive equations for the most usual freeway conditions. Whenever the situation fits within the limits of these formulas, it would increase accuracy to use this alternate method for calculating lane 1 volume rather than the freeway lane volume distribution curves. Nomographs (Figs. 8, 9, and 10) of some of these equations are presented for graphic solution of problems.

Vehicle Storage at On-Ramps

A secondary function performed by ramps is that of providing storage for cars interchanging between facilities. Although engineers endeavor to provide designs that will enable drivers to move without undue delay, traffic volumes often nullify this aim. Lack of adequate capacity at the ramp terminals can force the ramp to function as a storage area for varying periods of time. Stopped or slow-moving vehicles on a ramp are more an irritation than a major operational problem. However, if the available storage cannot absorb the excess demand, there is danger that the backed-up ramp vehicles will block through lanes on the interchanging highway.

As might be expected, interchanges vary considerably in their ability to cope with extreme traffic demands sufficiently well so that congestion is localized and not transmitted via the ramp to the other roadway. Most direct and semi-direct interchanges reported in this study had long ramps, usually two lanes in width, which provided adequate storage when needed. The same was true for the cloverleaf outer connections. Cloverleaf inner loops did suffer from inadequate storage capacity in some of the studies. One ramp which did not suffer from inadequate storage capacity, even though it carried 1,475 vph, was the inner loop from Cross Island Parkway southbound to the Long Island Expressway eastbound on Long Island, N.Y. This well-designed ramp has an auxiliary lane upstream from its exit from Cross Island Parkway and also at its entrance to Long Island Expressway. The ramp, 24 ft wide and fully two lanes operational throughout its length except at its merging end, has a minimum radius of 205 ft with 500-ft radii at its terminals. As shown in Figure 11, the two lanes narrow to one lane 14 ft wide at the merging end of the ramp. Fortunately, from the storage standpoint, if not the travel time standpoint, the ramp is longer than average (1,060 ft). During its peak hour of 1,475 vehicles, this ramp had several 5-min periods when the flow rate exceeded 1,700 vph merging into lane 1 of the Long Island Expressway. The expressway was carrying 3,900 vph in three lanes upstream from the ramp. The substantial storage (running room) afforded by the two ramp lanes localized the congestion which resulted when ramp vehicles were unable to merge fast enough to keep up with the heavy demand. Any design less liberal would have resulted in a backup into Cross Island Parkway, constricting its free-flowing traffic and producing hazardous maneuvers. It was decided to restudy this ramp, concentrating on determination of the number of vehicles traversing the ramp simultaneously and the average speed of the trailing vehicle while the ramp was emptying. During this study the ramp carried a peak of 1,512 vph. The "moving storage" checks, made on the average of once each 5-min period, ranged from 12 to 60 vehicles on the ramp simultaneously. At no time did the ramp vehicles back up into the Cross Island Parkway flow, although several times the

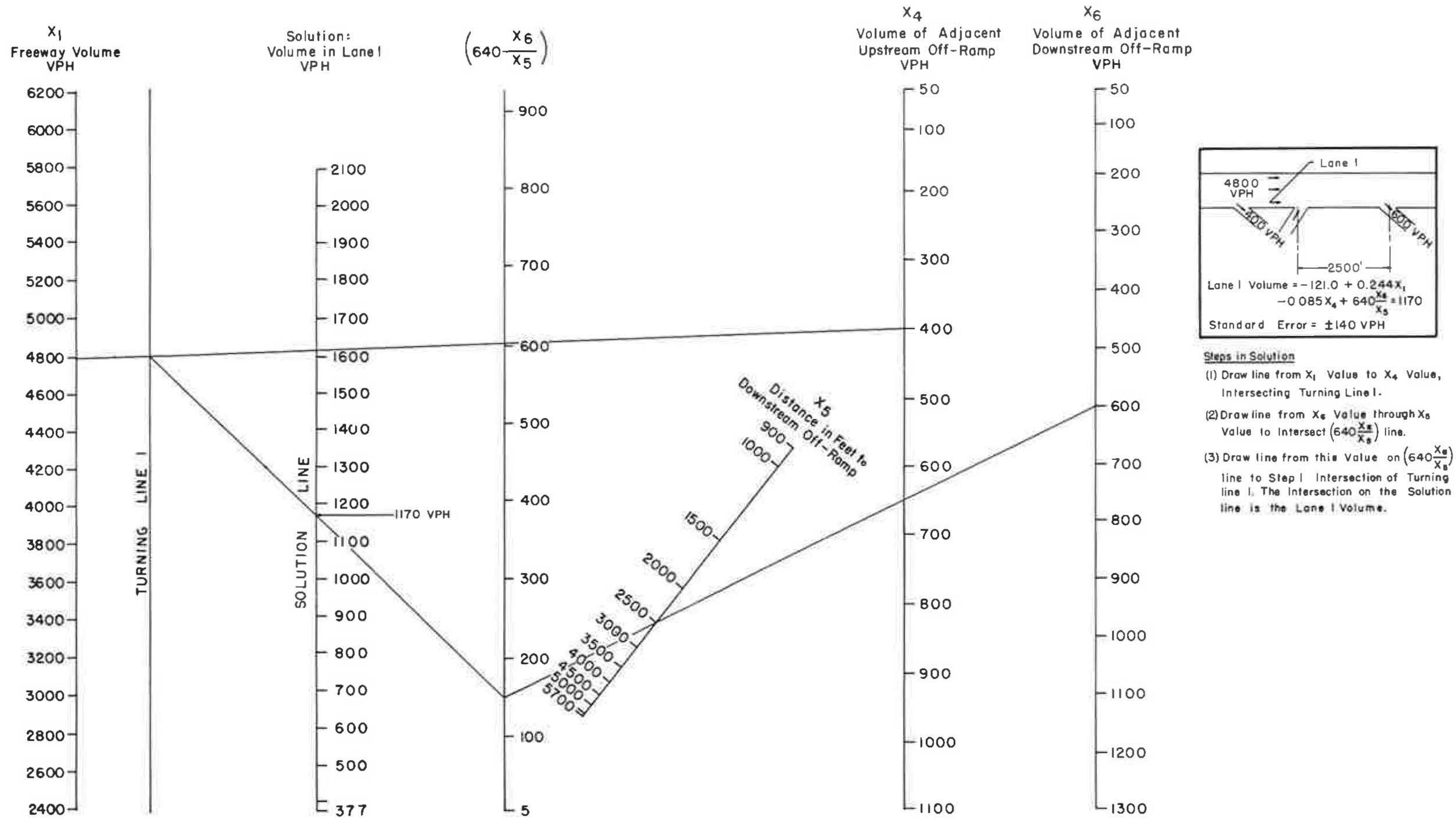


Figure 8. Nomograph for determination of lane 1 volume on 6-lane freeways.

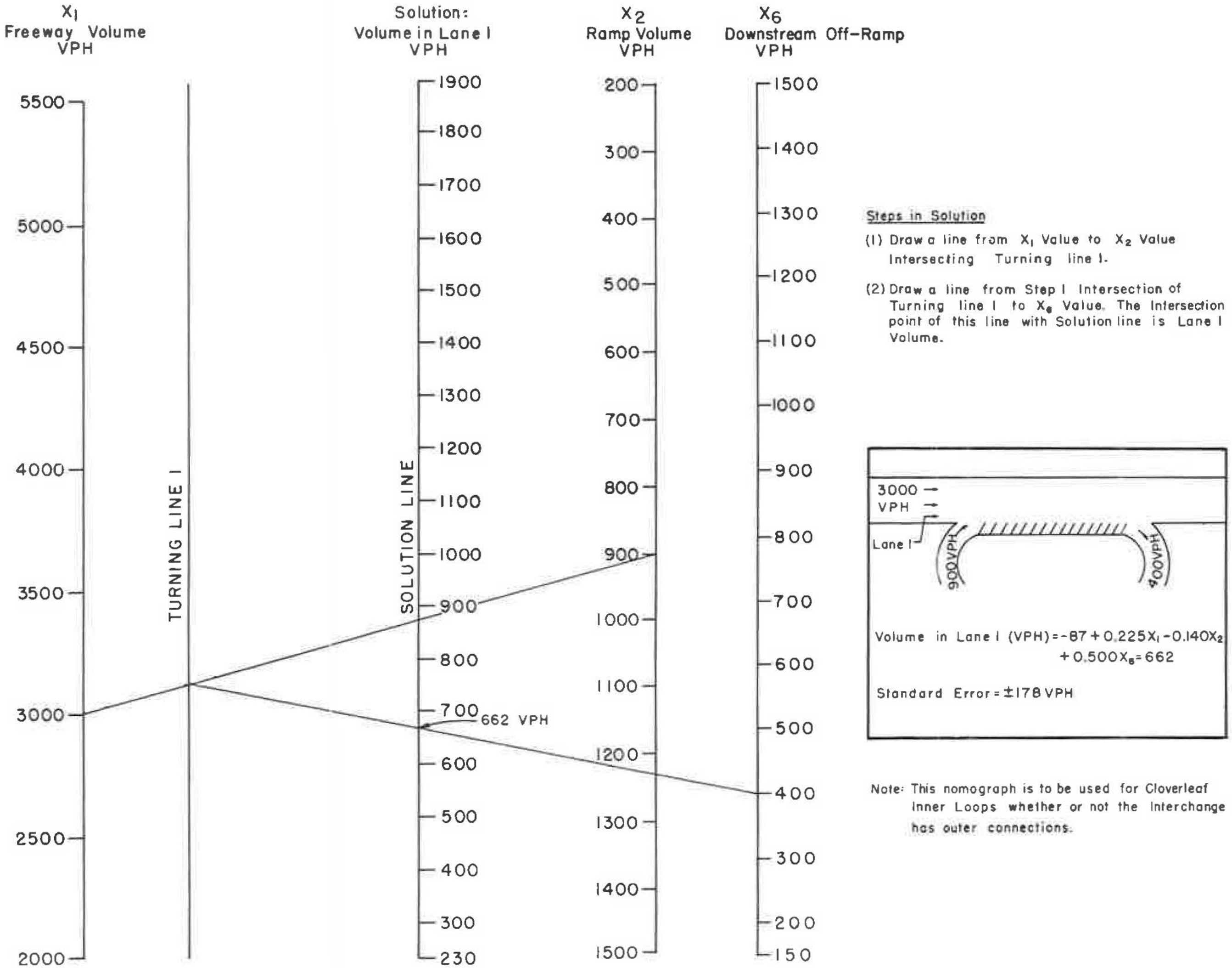


Figure 9. Nomograph for determination of lane 1 volume on 6-lane freeways at cloverleaf inner loops with auxiliary lane.

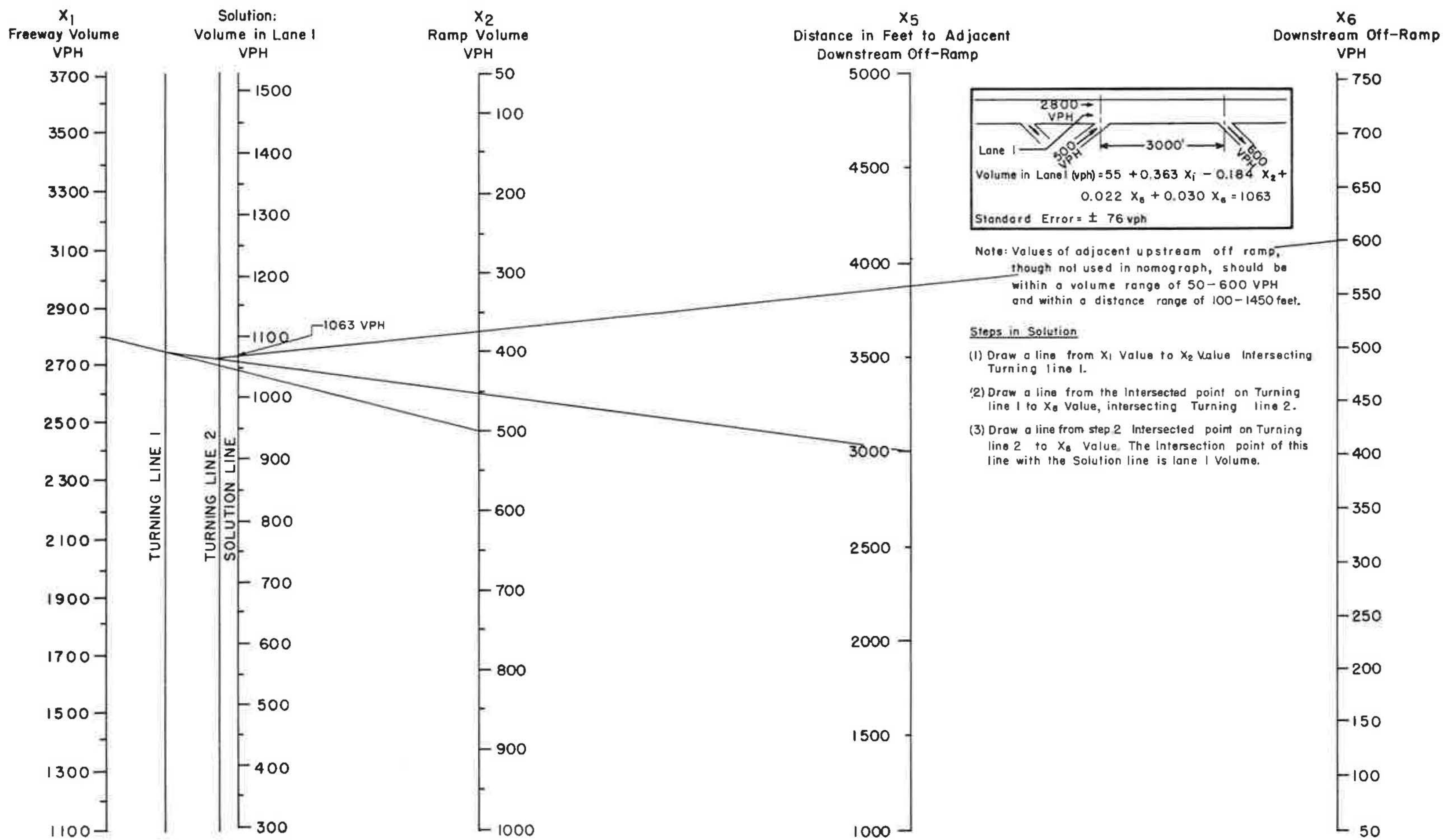


Figure 10. Nomograph for determination of lane 1 volume on 4-lane freeways.

Figure 11. Merging terminal of loop ramp from Cross Island Parkway southbound to Long Island Expressway eastbound on Long Island, N. Y.

ramp was completely full. Average speeds of the trailing vehicles while rounding the ramp were in the 8- to 21-mph range. During 39 min of this hour a vehicle was stalled halfway around the ramp on the inner lane, yet the ramp was able to function successfully during this period at a flow rate of 1,505 vph. The Long Island Expressway, into which the ramp vehicles had to merge, was moving slowly with some stop-and-go operation. The ramp vehicles (1,505-vph rate) had to weave through the off-ramp vehicles (777-vph rate) on the Long Island Expressway over the auxiliary lane distance of 625 ft. Obviously, traffic volumes on the Long Island Expressway were not conducive to free flow on connecting entrance ramps.

Figure 12. Beaubien ramp to Edsel Ford Expressway eastbound, showing poor operation resulting from 1-lane ramp being pressed into 2-lane service by crowding.

Slip ramps, which are usually very short, have a built-in disadvantage, especially where the traffic flow is moderate to heavy on both frontage road and freeway. Any congestion at the merging end can be quickly extended back onto the frontage road lanes.

At diamond on-ramps, storage capacity usually is not of much importance because signalization at the cross street controls the amount of traffic entering the ramp. However, in the case of a short ramp connecting directly to the cross street, vehicle storage can become critical if merging is difficult at the freeway end. This assumes a heavy slug of vehicles released to the ramp by the traffic signal. The data collected at the Beaubien on-ramp to the Edsel Ford Expressway eastbound in Detroit illustrate this situation.

At this location, a 600-ft long, 14-ft

wide (curb to curb) ramp with a 900-ft auxiliary lane operates smoothly at a rate of 900 vph, but when a heavy concentration of rush-hour traffic generated by the dense industrial development nearby delivers 20 to 40 vpm to the ramp, a chaotic situation develops. The stored vehicles on the ramp, waiting to merge, crowd into a two-lane operation with the outside lane using the auxiliary lane while the inside lane is forced to merge directly into an already heavily-loaded lane 1 (Fig. 12). Perhaps this illustrates the advantage of diamond ramps that come off frontage roads, giving the driver the option to continue along the frontage road if the ramp is overtaxed. Ramp vehicles may at times back up onto the frontage road, but this is not as serious as the disruption to the freeway when a one-lane ramp begins to operate as two lanes because of lack of storage room.

Ramp/Lane 1 Volume Proportions for Free-Flow Merge

The volume of traffic which can merge at a ramp-freeway connection is dependent on a number of variables associated with geometrics and traffic characteristics. One of these is the relative proportion of ramp and lane 1 volumes which are combined to make up the merge volume. One cannot expect the ease of merging to remain constant regardless of how the two volumes are distributed. For instance, where 1, 600 vehicles must merge and all other variables are held constant, it appears easier to merge 400 ramp vehicles with 1, 200 lane 1 vehicles than to merge 800 ramp vehicles with 800 lane 1 vehicles. Also, 1, 200 ramp vehicles can usually be merged with 400 lane 1 vehicles more readily than 800 ramp vehicles with 800 lane 1 vehicles. The merge volume in all instances is 1, 600 vph but the operation is considerably different in each of the three cases.

Figures 13 and 14 were developed in an effort to determine how the varying combinations of the two flows affect free-flow merge capacity. The curves are the least-squares fittings of ramp and lane 1 volumes for each category of ramp under free-flow merge conditions. The curves are derived from ramps with different geometrics, from freeways both new and old, and from cities of various sizes. No standardization to uniform conditions has been attempted. However, the ramps represented by a specific curve are of a certain category, such as diamonds or cloverleaf inner loops. This grouping provides a measure of uniformity, even though combinations of ramps with different geometrics are necessary to create workable samples. For instance, a diamond ramp having a 700-ft acceleration lane is in the same grouping as a diamond ramp having no acceleration lane. The result is a curve giving a broad average of conditions and reflecting the relative capability of the different type interchanges within these average conditions.

Usually the lane 1 volume is taken as the independent variable in making the least squares calculations, as shown in Figure 13. Strictly speaking, however, lane 1 is not completely independent of the ramp. Rather, the free-flow merge is somewhat of an interaction between the two traffic streams. Then why not use the ramp as the independent variable and lane 1 as the dependent variable? This is done in Figure 14. Arguments can be presented for both cases. The problem is that given the same set of data, the least squares solution of the best curve fit will give different answers for free-flow merge, depending on which flow is taken as the independent variable. As an example, given a cloverleaf outer connection on-ramp with freeway lane 1 volume of 600 vph, how many ramp vehicles can be accommodated while maintaining free-flow merge? Figure 13 shows 800 ramp vehicles to be the answer, whereas Figure 14 would give 1, 000 ramp vehicles.

It is the intention in this progress report to show both sets of curves and present the arguments for each. Using lane 1 volume as the independent variable (Fig. 13) seems most logical for several reasons, as follows:

1. Lane 1 generally has the right-of-way.
2. Lane 1 speeds are steadier than those on the ramp.
3. Ramp vehicle drivers generally make the bulk of the merging decisions and the makeup of the lane 1 traffic largely determines how and where the merge is accomplished. Ramp performance appears to be dependent on the lane 1 volume.

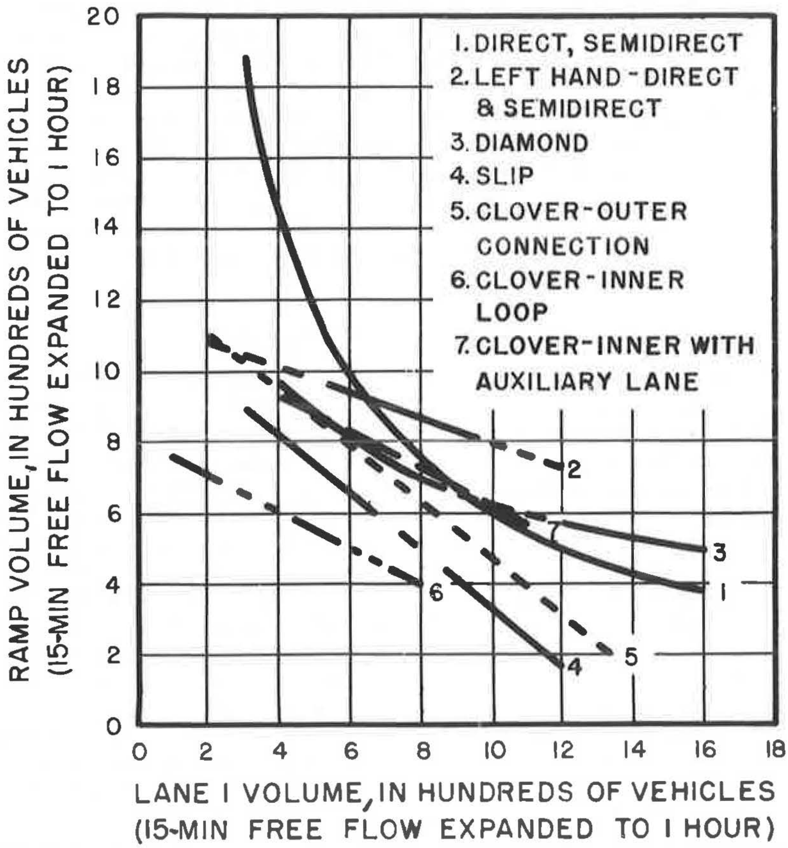


Figure 13. Distribution of ramp and lane 1 volumes for free-flow merge (lane 1 as independent variable).

A less forceful case could be presented for using the ramp as the independent variable (Fig. 14), as follows:

1. The ramp vehicle driver has a maneuver to perform—merging. There is no possible deviation from this goal. The lane 1 vehicle driver, on the other hand, can usually deviate from his path by shifting to lane 2 as a consequence of ramp pressure.
2. High-volume ramps can dominate the merge, forcing lane 1 vehicles to slow down or adjust speeds to those of the merging vehicles.
3. Some lane 1 drivers adjust their speeds to accommodate merging ramp vehicles regardless of the hourly ramp volume or the pressure exerted by the ramp vehicles.

The author prefers the argument in favor of lane 1 as the independent variable for application of the least squares solution. The equations for the curves in Figure 13 are given in Table 1.

As can be ascertained from the standard errors given in Table 1, there is a rather large spread in the data within each ramp category. This is not unexpected because, as mentioned previously, the geometrics and traffic characteristics for ramps within each category varied considerably.

The exponential curves of Figure 13, which are used for direct, semidirect, and diamond ramps, are plotted separately in Figures 15, and 16, together with the upper and lower limits of the standard errors of estimate. The standard error of estimate for an exponential curve is not a constant value, but is a constant percentage above and below the curve value. For instance, the limits of the standard error of estimate shown in Figure 16 for diamond ramps are 34.3 percent above and 25.6 percent below

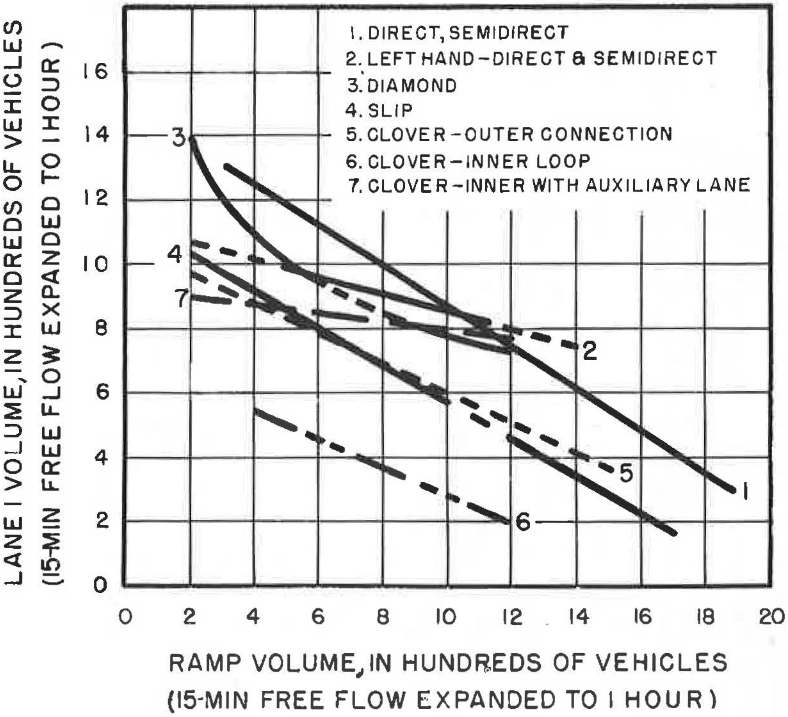


Figure 14. Distribution of ramp and lane 1 volumes for free-flow merge, by ramp type (ramp as independent variable).

the curve value. These percentages apply to the ramp volume, which is added to the given freeway lane 1 volume to form the free-flow merge volume. For a high lane 1 volume combined with a low ramp volume, the possible variation of ramp volume within the standard error of estimate would be quite low. However, for a low lane 1 volume there would be a larger variation in the ramp volume that could be accommodated in the free-flow merge.

Examination of Figure 13 discloses that for most ramp and lane 1 volumes, direct and semidirect ramp connections have the highest free-flow merge volumes, followed in order by diamonds, cloverleaf inner loops with auxiliary lanes, cloverleaf outer

TABLE 1

RAMP VOLUME FORMULAS

Ramp Type	Curve Fit	Equation for Free-Flow Ramp Vol. (vph)	Std. Error of Estimate (vph)
Direct, semidirect	Exponential	$R = 436,909 \text{ (vol. lane 1)}^{-0.955}$	—[a]
Left-hand, direct, semi-direct	Straight line	R = 1,153 - 0.35 (vol. median lane)	336
Diamond	Exponential	$R = 17,029 \text{ (vol. lane 1)}^{-0.479}$	—[b]
Slip	Straight line	R = 1,143 - 0.82 (vol. lane 1)	312
Clover, outer connection	Straight line	R = 1,257 - 0.79 (vol. lane 1)	318
Clover, inner loop	Straight line	R = 805 - 0.51 (vol. lane 1)	273
Clover, inner loop with auxiliary lane	Straight line	R = 1,139 - 0.52 (vol. lane 1)	329

[a] See Figure 15.
[b] See Figure 16.

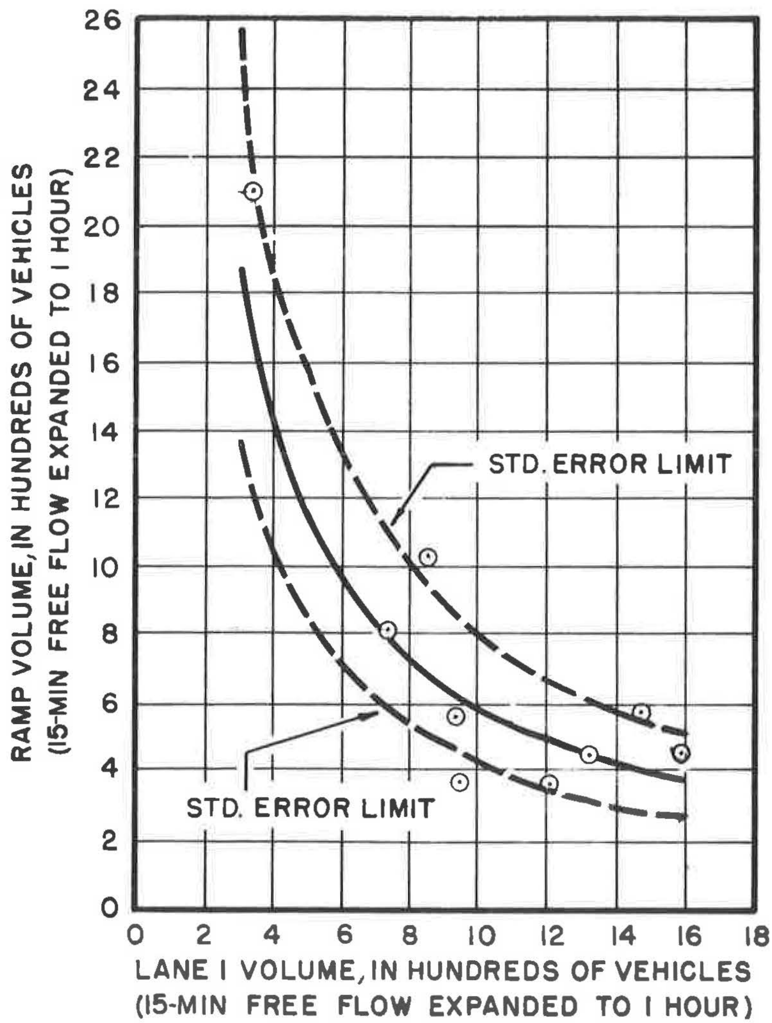


Figure 15. Distribution of ramp and lane 1 volumes for free-flow merge for direct and semidirect ramps (lane 1 as independent variable).

connections, slip ramps, and cloverleaf inner loops without auxiliary lanes. Figure 14 shows much the same order, except that slip ramps and cloverleaf outer connections are nearly identical in merging capacity.

The two sets of curves differ markedly, however, in identifying the optimum proportion for ramp and lane 1 volumes. Figure 13, with lane 1 independent, indicates that the highest free-flow merges can be expected when ramp volumes are low and lane 1 volumes are high, except for direct and semidirect ramps. Figure 14, with the ramp independent, favors high ramp volumes and low lane 1 volumes for highest free-flow merge. This appears logical enough when it is remembered that the ramp is considered independent and at high volumes tends to dominate lane 1 traffic. Finally, the exponential curve shown in Figure 13 and also in Figure 15 for direct and semidirect ramps indicates that when either flow is dominant the free-flow merge volumes will be higher than when the two flows approximate each other in volume. Figures 17 and 18, showing the Route 22 westbound connection to the Garden State Parkway southbound in New Jersey, illustrate a location studied where a heavy ramp (1,800 vph) dominated a merge of 2,100 vph. The location was free flowing, primarily because of the light parkway volume, absence of commercial vehicles, and excellent geometrics. As

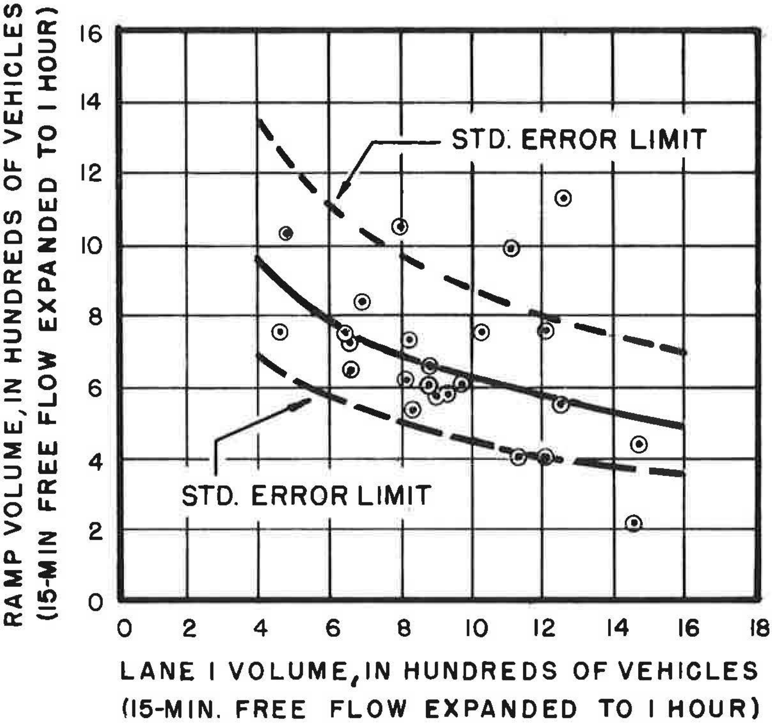


Figure 16. Distribution of ramp and lane 1 volumes for free-flow merge for diamond ramps (lane 1 as independent variable).

Figure 17. Route 22 ramp to Garden State Parkway southbound in New Jersey.

stated previously, the author prefers to treat lane 1 as the independent variable and the ramp as dependent.

When use is made of these curves, lane 1 volume at a given freeway volume can be determined by reference to Figures 3, 4, 5, 6, and 7. The ramp volume determined

Figure 18. Looking back toward merge of Route 22 ramp with Garden State Parkway southbound in New Jersey.

from either Figure 13 or Figure 14 will then represent an average condition for the ramp category. If more accuracy is desired, it would be wiser (though more time-consuming) to apply the regression analysis formulas described in the following section.

Free-Flow Merge for One-Lane On-Ramps

Two formulas were developed by regression analysis for use in computing free-flow merge at 1-lane right-hand on-ramp connections. Appendix A gives a detailed discussion of the variables used and their relative effect on capacity calculations. Table 4 (Appendix A) presents details of the regression analysis.

The first formula, derived from 73 observations at all types of interchanges combined, can be applied to all types of interchanges except left-hand connections. Data were insufficient to permit development of a formula for left-hand ramps. This general formula for one-lane right-hand ramps at all types of interchanges is

$$\text{Free-flow merge (vph)} = 528 + 8.5X_1 - 16.5X_2 + 7.6X_3 - 1.0X_4 + 0.22X_5 + 0.071X_6 \quad \text{(A)}$$

The second formula was derived after deleting the very short ramps and ramps of sharp curvature near the nose (slip ramps and cloverleaf inner loops) from the 73 observations, leaving a remainder of 55 observations. This formula

$$\text{Free-flow merge (vph)} = 441 + 10.0X_1 - 18.0X_2 + 9.5X_3 - 5.0X_4 + 0.014X_5 + 0.068X_6 \quad \text{(B)}$$

should be used only for one-lane right-hand ramps of the following types: Diamond, semidirect, direct, trumpet outer connection, and cloverleaf outer connection. If used erroneously for other types, such as cloverleaf inner loops, it will give values that are too low.

Several of the coefficients in the two formulas differ slightly from the calculated coefficients given in Table 4 (Appendix A). Those differing have been rounded slightly to facilitate computation. This rounding does not affect any free-flow merge computation by more than a few vehicles.

In these formulas the variables are:

X_1 = % freeway utilization. This is a measure of the freeway use immediately upstream from the on-ramp nose. It is the hourly freeway volume (or 15-min free flow expanded to 1 hour) divided by the number of freeway lanes multiplied by 2,000 vph possible capacity per lane, or

$$\% \text{ Freeway utilization } = \frac{\text{Freeway volume (vph)}}{\text{No. of lanes } \times \text{ 2,000 vph/lane}} \times 100 \qquad \text{(C)}$$

X_2 = % commercial vehicles in the merge. This is the number of commercial vehicles in the merge (ramp + lane 1) divided by the expected number of vehicles in the merge, or

$$\% \text{ c.v. in merge } = \frac{\text{c.v. (Ramp + Lane 1) vph}}{\text{Expected merge volume (vph)}} \times 100 \qquad \text{(D)}$$

X_3 = ramp/merge ratio. This is a measure of the merge components, consisting of the ramp volume divided by the merge volume, or

$$\text{Ramp/Merge ratio } = \frac{\text{Ramp volume (vph)}}{\text{Expected merge (Ramp + Lane 1) volume (vph)}} \times 100 \qquad \text{(E)}$$

X_4 = angle of convergence, in degrees. This is the interior angle made between the right edge of lane 1 and the left edge of the ramp at right-hand on-ramps. (The glossary gives details on measuring this angle when the ramp and/or freeway is curved.)

X_5 = length of acceleration lane, in feet.

X_6 = metropolitan area population, in 1,000's. (This value should not exceed 5,000 as applied to the formula.)

In using the formulas, whole numbers and not decimal equivalents should be used for the percentages expressed in X_1, X_2, and X_3 (i.e., for 27 percent use 27, not 0.27). The metropolitan area population, X_6, should be obtained from the 1960 census, keeping in mind that for metropolitan area populations larger than 5,000,000, the figure 5,000 should be used.

The results of the two formulas, broken down by ramp types, are compared in Table 2. The formula based on 73 observations generally predicts higher values for cloverleaf outer connections and lower values for diamond ramps than does the other. All other ramp types are grouped together, with similar results for the two formulas. Everthing considered, any difference in results between the two formulas is minor and for simplicity the formulas based on 73 observations should be used. The other formula, although more limited as the ramp types represented, has the advantage of a lower standard error of estimate.

It is interesting to note that in Table 4 (Appendix A) the mean value of the free-flow merge is 1,569 vph for the 73 observations. This is a close approximation of the 1,500 vph/lane assigned as urban practical capacity for freeways. The standard deviation of 288 vph indicates quite well that there is no magic number which can be used as

TABLE 2

COMPARISON OF RESULTS FROM FORMULAS BASED ON 73 AND 55 OBSERVATIONS

Ramp Type	Observations	Free-Flow Merge (vph)			Difference in Means[a]
		True Mean	Predicted Mean		
			73 Obs.	55 Obs.	
Clover, outer	17	1,502	1,545	1,503	+42
Diamond	27	1,645	1,621	1,655	-34
Direct; semidirect; and trumpet, outer	11	1,724	1,684	1,695	- 9

[a]73 observations formula compared with 55 observation formula.

a merge capacity figure for general application. Of the 73 observations, 51 (70%) fall within the 1,281- to 1,857-vph range of one standard deviation. This appears to be a rather large range but, once again, it should be remembered that the data used are not only from different sections of the nation but also represent a wide range of design and traffic conditions. The variation in merge capacity within a given more or less homogeneous system should be considerably less.

Also of interest are the means of the freeway volumes upstream and downstream from the ramp for the 73 observations. Upstream from the ramp (before merge) the mean of the freeway volumes was 9.5 percent below practical capacity. The mean of the ramp volumes was 597 vph. Downstream (after merge) the mean of the freeway volumes was 4.8 percent above practical capacity.

Figure 19 presents a means of determining the percentage of commercial vehicles in lane 1 of the freeway at the ramp nose. A word of caution is needed regarding its use. Partially because of local laws, there is much variation between cities in the truck distribution among lanes. If local data are available, it would increase the accuracy to use them rather than Figure 19 when applying the formulas.

The formulas can be used for a number of purposes. Used in conjunction with lane volume distribution curves or lane 1 volume equations and commercial vehicle distribution (Fig. 19), the formulas provide a much needed capacity computation tool. Also, if some traffic counts are made, facilities already in operation can be evaluated for quality of performance. The formulas could be very useful where possible ramp closures or monitoring are being evaluated in the hope of maintaining a free-flow volume level on congested freeways. Another possible use is the prediction of future trouble spots as traffic volumes increase on newly opened networks.

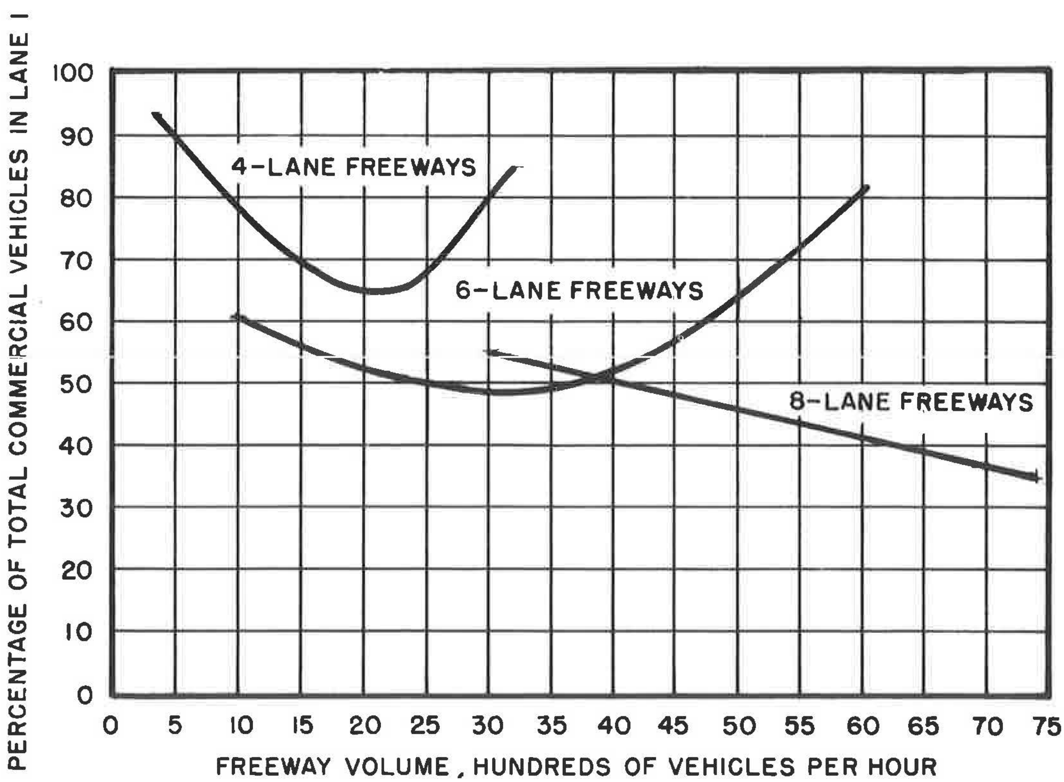


Figure 19. Percentage of total commercial vehicles in lane 1 of 4-, 6-, and 8-lane freeways immediately upstream from on-ramp entrances.

Sample Problem No. 1

Given: A semidirect interchange along a 6-lane freeway in Detroit, Mich. The freeway carries 4,300 vph just upstream from the ramp nose. Ramp volume is 800 vph. Commercial vehicles make up 3 percent of the freeway traffic and 3 percent of the ramp traffic. The angle of convergence is 10° and the length of acceleration lane is 600 ft.

Find:

1. The "expected merge" for this on-ramp connection under the given traffic volumes.
2. The predicted "f.f. merge" using the formula derived from 55 observations.
3. Adequacy of design without considering the standard error of estimate.
4. Adequacy of design considering the standard error of estimate given in Table 4.

Solution:

Using Figure 5, lane distribution for 6-lane freeways, 21.5 percent of the freeway stream will be in lane 1, and land 1 (vph) = 0.215 × 4,300 = 925.

"Expected merge" = 925 vph (lane 1) + 800 vph (ramp) = 1,725 vph.

Number of commercial vehicles in freeway stream = 0.03 × 4,300 = 129.

Number of commercial vehicles in ramp traffic = 0.03 × 800 = 24.

Using Figure 19 for commercial vehicle distribution, 55 percent of the 129 freeway commercial vehicles will be in lane 1, or commercial vehicles in lane 1 = 0.55 × 129 = 71.

Using the 55-obs. formula:

Plus quantities for formula

Constant = 441

$$X_1,\ \%\ \text{fwy. util.} = \frac{4{,}300}{3 \times 2{,}000\ \text{vph/lane}} \times 100 = 72$$

$$X_3,\ \text{ramp/merge ratio} = \frac{800}{925 + 800} \times 100 = 46$$

X_5, acceleration lane = 600

X_6, metropolitan area pop. = 3,762 (from 1960 census) (1,000's)

Minus quantities for formula

$$X_2,\ \%\ \text{c.v. in merge} = \frac{71 + 24}{925 + 800} \times 100 = 5.5$$

X_4, angle of convergence = 10°

Applying the 55 obs. formula:

$$\text{Free-flow merge} = 444 + 10.0(72) - 18.0(5.5) + 9.5(46) - 5.0(10) + 0.14(600) + 0.068\,(3{,}762) = 1{,}789\ \text{vph}.$$

Inasmuch as the 1,789-vph free-flow merge predicted by the formula is more than the "expected merge" of 1,725 vph, the facility can be considered adequate. However, if the design standard is to keep the free-flow merge within the standard error of estimate, the designer would have to consider the minimum free-flow merge within the standard error of estimate, which is 1,630 vph (1,789 vph prediction - 159 vph standard error of estimate). The probability of having a free-flow merge of more than 1,630 vph is approximately 0.84. The 1,630 vph is less than the "expected merge," so adjustments would have to be made in the design.

Answers:

1. 1,725 vph "expected merge."

2. 1,789 vph predicted free-flow merge.
3. Design is adequate without taking into consideration the standard error of estimate.
4. Design is inadequate if standard error of estimate is applied.

Those using these formulas should understand the relationship between the "expected merge" and the "free-flow merge." The "expected merge" is the merge which is forecast, taking into account the ramp and freeway volumes. If it is less than the computed "free-flow merge," the facility will operate satisfactorily and the merge taking place will be the "expected merge." However, if the "expected merge" is higher than the "free-flow merge," the indication is that the operation will be congested because more vehicles will be attempting to merge than can be accommodated in a satisfactory manner by the facility.

By itself, the free-flow merge volume has limited significance. The facility can be operating very well with a low free-flow merge volume, provided the "expected merge" is even lower. Increasing the freeway and/or ramp volumes would increase the computed "free-flow merge" but the "expected merge" would increase even more rapidly so that congested operation would soon result.

Different "expected merge" and "free-flow merge" volumes will result if the proportion of the ramp and freeway volumes are varied while keeping a constant total volume. For instance, using the sample problem condition and varying the volumes up and down by 200-vph increments so that computations are made for 600 ramp vehicles merging with 4,500 freeway vehicles and 1,000 ramp vehicles merging with 4,100 freeway vehicles, the results are as follows:

Free-Flow Volume (vph)				
Total	Freeway	Ramp	Expected Merge	Predicted Free-Flow Merge
5,100	4,100	1,000	1,861	1,835[a]
5,100	4,300	800	1,725	1,789
5,100	4,500	600	1,568	1,734

[a]Congestion predicted.

The foregoing comparison shows that as the ramp volume increases, the "expected merge" and the "free-flow merge" volumes both increase, but the former much more rapidly. In the case of 1,000 ramp vehicles merging into a freeway stream of 4,100 vehicles, some congestion can be expected because the "free-flow merge" is less than the "expected merge." Much as experience with freeway operation might lead one to assume, the best operation of the three cases cited is when 600 ramp vehicles merge into a freeway stream of 4,500 vehicles.

Alternative Method for Computation of Free-Flow Merge

The formulas used in the preceding section depended on a computation of lane 1 volume by the use of curves set up for varying freeway volumes (Figs. 3, 4, 5, 6, 7). These least squares curves represent the best fit for the lane use data obtained in this study. Indirectly, the curves reflect study ramp pressure on lane 1 volumes, adjacent ramp action on lane 1 volumes, and the effects of the various other components (such as signing and geographical location) which are determinants in the use of lane 1. These curves are more accurate at high freeway volumes than at volumes below practical capacity. At these lower volume levels there is more margin for error as local conditions (such as location and volume of adjacent ramps) exert more of an influence on the freeway volume distribution.

In an attempt to more closely fit the conditions at hand and narrow the margin of error, five equations have been developed by multiple regression analysis for use in calculating the lane 1 volumes used in the free-flow merge calculations. These equations take into account the distances to and volumes of adjacent upstream and downstream off-ramps, as well as the freeway and ramp volumes at the connection for which computations are being made. Unfortunately, there were insufficient data to permit derivation of equations for conditions other than the most common possibilities. Table 3, as well as Appendix B, gives the conditions for which the equations are applicable. Those using the equations should not extrapolate or use values outside the ranges shown in Table 3. For those who prefer graphic solutions, nomographs (Figs. 8, 9, and 10) are given for Eqs. 1, 4, and 5.

The equations and the broad requirements for the use of each are as follows:

Equation No. 1—

Condition: For 6-lane freeways when the on-ramp under consideration is bracketed by adjacent off-ramps, upstream and downstream, and no auxiliary lane connection exists to the adjacent downstream off-ramp.

$$\text{Volume in lane 1 (vph)} = -121 + 0.244\,(\text{freeway volume in vph}) - 0.085\,(\text{volume of adjacent upstream off-ramp in vph}) + 640\,\frac{(\text{volume of adjacent downstream off-ramp in vph})}{(\text{distance, in feet, to adjacent downstream off-ramp})}$$

Equation No. 2—

Condition: For 6-lane freeways when the on-ramp under consideration has an adjacent upstream off-ramp and is connected to an adjacent off-ramp less than 1,000 ft downstream by an auxiliary lane.

$$\text{Volume in lane 1 (vph)} = 62 + 0.232\,(\text{freeway volume in vph}) - 0.072\,(\text{ramp volume in vph}) - 0.041\,(\text{length, in feet, of auxiliary lane}) + 0.432\,(\text{volume of adjacent downstream off-ramp in vph})$$

Equation No. 3—

Condition: For 6-lane freeways when the on-ramp under consideration is connected to an adjacent off-ramp less than 1,000 ft downstream by an auxiliary lane, and there is either no nearby upstream ramp or, if so, its volume is negligible.

$$\text{Volume in lane 1 (vph)} = -162 + 0.273\,(\text{freeway volume in vph}) - 0.195\,(\text{ramp volume in vph}) + 0.635\,(\text{volume of adjacent downstream off-ramp in vph})$$

Equation No. 4—

Condition: For 6-lane freeways at cloverleaf interchanges where the inner loop on-ramps is connected to the inner loop off-ramp by an auxiliary lane. The interchange may or may not have outer connections. The equation does not require them and applies only to inner loop on-ramps.

$$\text{Volume in lane 1 (vph)} = -87 + 0.225\,(\text{freeway volume in vph}) - 0.140\,(\text{ramp volume in vph}) + 0.500\,(\text{volume of adjacent downstream inner loop off-ramp in vph})$$

Equation No. 5—

Condition: For 4-lane freeways when the on-ramp under consideration is bracketed by adjacent off-ramps, upstream and downstream, and no auxiliary lane connection exists to the adjacent downstream off-ramp.

$$\text{Volume in lane 1 (vph)} = 55 + 0.363\,(\text{freeway volume in vph}) - 0.184\,(\text{ramp volume in vph}) + 0.022\,(\text{distance in feet to adjacent downstream off-ramp}) + 0.030\,(\text{volume of adjacent downstream off-ramp in vph})$$

Sketches of these layouts are shown in Table 5 (Appendix B), together with the equations and associated statistical data.

TABLE 3

REQUIRED RANGES OF VARIABLES FOR VALID USE OF EQUATIONS

Eq.	Volume (vph)		Adj. Upstream Off-Ramp		Adj. Downstream Off-Ramp	
	Freeway	Ramp	Distance (ft)	Volume (vph)	Distance (ft)	Volume (vph)
1	2,400 - 6,200	100 - 1,700	900 - 2,600	50 - 1,100	900 - 5,700	50 - 1,300
2	1,900 - 6,200	150 - 1,900	450 - 2,150	50 - 1,000	550 - 950	50 - 1,000
3	1,900 - 6,200	50 - 1,900	-	-	550 - 950	50 - 1,000
4	2,000 - 5,600	200 - 1,500	-	-	450 - 850	150 - 1,500
5	1,100 - 3,700	50 - 1,000	100 - 1,450	50 - 600	1,000 - 5,000	50 - 750

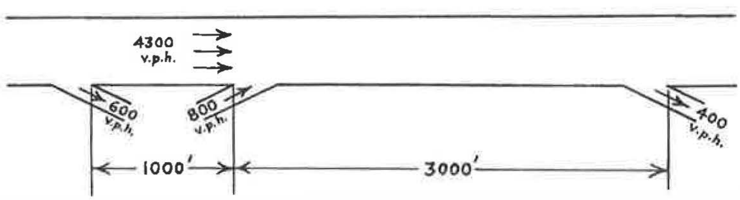
Those who use the equations should not be confused because some of the required validating conditions are not found as variables in the equations. For example, in Eq. 2 an adjacent off-ramp is required between 450 and 2,150 ft upstream from the ramp under consideration. However, the equation does not contain variables relating to this specified adjacent upstream off-ramp. These variables are missing from the equation because their effect was found to be negligible and so their input was deleted in the derivation of the formula. Nevertheless, conditions other than those specified might cause a different freeway lane volume distribution leading to an erroneous calculation.

Sample Problems Using Lane 1 Volume Equations

To understand the use of the equations, several sample problems will be worked:

Sample Problem No. 1A—

Given: The same conditions as in Sample Problem No. 1, the only change being that adjacent upstream and downstream off-ramp conditions are also given, as follows:



The adjacent upstream off-ramp, 1,000 ft away, carries 600 vph; the adjacent downstream off-ramp, 3,000 ft away, carries 400 vph.

Find:

1. The "expected merge" for this on-ramp connection under the given traffic volumes using a lane 1 volume equation.
2. The predicted free-flow merge, using the formula derived from 55 observations.

Solution: The given conditions fall within the requirements for use of Equation 1, so this equation is used to calculate lane 1 volume.

$$\text{Volume in lane 1 (vph)} = -121 + 0.244\,(4,300) - 0.085\,(600) + 640\,\frac{(400)}{(3,000)} = 962$$

$$\text{Expected merge} = 962 \text{ vph (lane 1)} + 800 \text{ vph (ramp)} = 1,762 \text{ vph}$$

The free-flow merge formula from 55 observations would then be applied as in Sample Problem No. 1 and, because the "expected merge" of 1,762 vph is less than the predicted free-flow merge of 1,782 vph, there should be no congestion. This does not take into account the possible application of the standard errors for the lane 1 volume or the free-flow merge. This aspect is discussed later.

The foregoing answers exhibit little difference from those calculated for Sample Problem No. 1. The most likely reason for the close approximation is that the adjacent ramp conditions used in the equation calculation of lane 1 volume were quite ordinary or average. The difference in the two methods of calculating lane 1 volume becomes more apparent if the adjacent downstream off-ramp carried 700 instead of 400 vph.

Using Equation No. 1, the lane 1 volume would now be 1,026 vph, the "expected merge" 1,826, and the predicted free-flow merge (using the formula from 55 observations) 1,785 vph. The presence of the rather heavy downstream off-ramp, now carrying 700 vph, means more freeway vehicles, in anticipation of exiting, will be using lane 1, thus raising the "expected merge." The "expected merge" now exceeds the free-flow merge so congestion is predicted. The addition of 300 more vph exiting downstream has changed the forecast from free flow to congestion.

If, as before, calculations are also made for 1,000 ramp vph merging into a freeway stream of 4,100 vph and 600 ramp vph merging into a freeway stream of 4,500 vph, using the lane 1 volume formulas which take into account the adjacent ramps, the values are as follows:

Free-Flow Volume (vph)						
			Expected Merge		Predicted Free-Flow Merge	
Total	Freeway	Ramp	400 Vph Exiting Downstream	700 Vph Exiting Downstream	400 Vph Exiting Downstream	700 Vph Exiting Downstream
5,100	4,100	1,000	1,913	1,977	1,815[a]	1,810[a]
5,100	4,300	800	1,762	1,826	1,782	1,775[a]
5,100	4,500	600	1,611	1,675	1,727	1,720

[a]Congestion predicted.

These calculations should be compared with those given earlier.

As more data become available, other equations can be developed to include other adjacent ramp conditions, such as an adjacent on-ramp upstream from the on-ramp under consideration. It is doubtful that equations can be developed which make use of upstream and downstream ramps other than the immediately adjacent ramps. Considerable data are needed to allow accomplishment of such a task. In the interim, if the equations are not applicable to the given freeway layout and volumes, the lane volume curves (Figs. 3, 4, 5, 6, and 7) can be used to determine lane 1 volume just upstream from the ramp nose.

No mention has been made of how to handle the standard error of estimate for a lane 1 volume equation as given in Table 5 (Appendix B). If the objective is to reduce the risk of a failure (i.e., traffic congestion), the logical procedure would be to add the standard error to the calculated lane 1 volume. This would have but a slight effect on the free-flow merge calculation, but it would increase the "expected merge" by the amount of the standard error, thereby decreasing relatively the margin between the "expected merge" and the predicted free-flow merge, for free-flowing conditions.

It should be remembered, of course, that there is also a standard error for the free-flow merge calculation. The two standard errors involved in the two calculations (lane 1 volume and free-flow merge) might be additive in the direction of poorest performance or additive in the direction of optimum performance, or the standard errors might tend to cancel each other out. In the case of operational-type problems, such as ramp closure decisions on congested freeways, it seems sufficient to apply only the standard error of the free-flow merge equation. Subtracting this standard error gives the user a free-flow merge value (a lower limit) which will be exceeded in actual freeway operation approximately 84 percent of the time. In design-type problems, however, some may feel it prudent also to apply the standard error of the lane 1 volume equation. The calculated lane 1 volume increased by the amount of the standard error would then be used in the free-flow merge calculation. The additional measure of safety provided by applying the lane 1 standard error cannot readily be measured in terms of the resulting free-flow merge, but overall the measure of statistical confidence would now be comparable with that usually used in research work—a confidence interval exceeding 90 percent.

Two-Lane On-Ramps

Some ramps designed as 2-lane facilities operate instead as 1-lane, either through lack of demand or because of geometric conditions which make driving them single file more comfortable. Given sufficient demand, operation will become 2-lane. Even 1-lane ramps sometimes operate as 2-lane facilities when high demand forces ramp drivers to double up, as already discussed for the Beaubien ramp in Detroit. For purposes of this report, 2-lane ramps are those designed and operating as such at the terminal of the ramp.

One of the most interesting 2-lane ramps studied was the Northern State Parkway semidirect connection to the Long Island Expressway westbound. This location was studied four times with "peak merge hour" ramp volumes ranging from 2,040 to 2,265 vph. Inasmuch as the Long Island Expressway was opened to a point only 0.6 mi east (upstream) of this ramp at the time of the first two studies, total peak-hour freeway volume for three lanes was only 1,000 to 1,500 vehicles. A few weeks later, five more miles of expressway were opened with peak-hour freeway volume upstream from the ramp of 2,735 vehicles in the morning peak and 1,949 vehicles in the afternoon peak. Operation throughout the peak periods was mostly free-flowing, with only a few slowdowns. Free-flow merging volumes were 2,444, 2,700, 2,468, and 2,688 vph as shown in the following: The 15-min f.f. volumes expanded to 1 hour were as follows:

Study		Free-Flow Volume[a] (vph)							
		Ramp			Freeway				
No.	Time	Lane B	Lane A	Merge[b]	Lane 1	Lane 2	Lane 3	Total	Total[c]
1	A.M.	620	1,376	2,444	448	692	532	1,672	3,668
2	P.M.	1,332	1,160	2,700	208	412	380	1,000	3,492
3	A.M.	684	1,384	2,468	220	1,108	1,076	2,404	4,652
4	P.M.	876	1,528	2,688	284	816	848	1,948	4,352

[a]15-min f.f. volume expanded to 1 hour. [b]Ramp B + Ramp A + Lane 1.
[c]Freeway + Ramp.

The comparatively low freeway volumes permitted ramp lane A traffic to merge directly into lane 2 on occasion. Although the merge volumes given are the additions of the ramp lanes and lane 1, recognition should be given to this role that lane 2 plays when freeway volumes are at low levels. The high merge figures are more understandable when the low-volume nature of the freeway traffic is understood. An acceleration lane 810 ft long was used by the ramp lane B vehicles. In three of the four studies, lane A carried considerably more traffic than lane B. The overall ramp lane volume percentages at ramp volume levels between 1,400 and 2,600 vph are shown in Figure 20 under N.Y.-36a, b, c, d. Evidently, the drivers preferred to take their chances

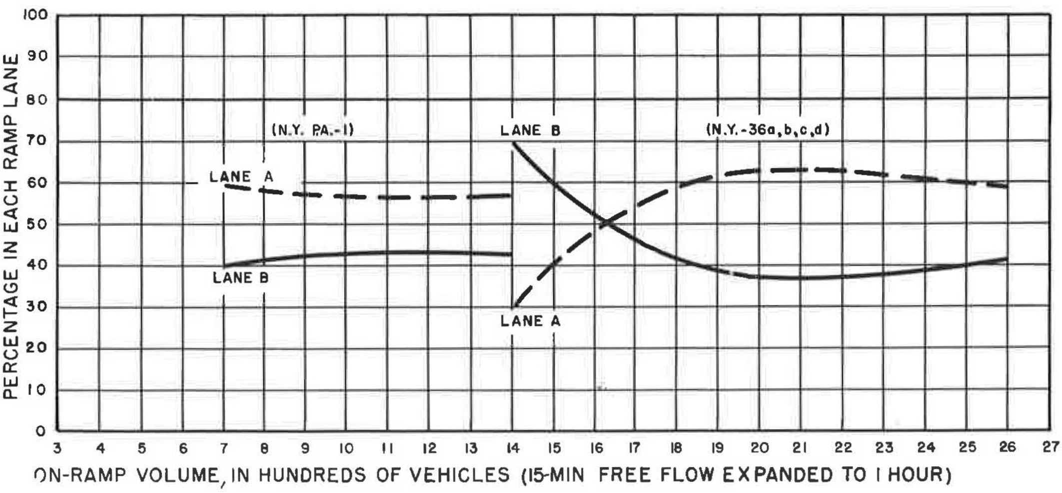


Figure 20. Ramp volume distribution for 2-lane on-ramps (two locations).

in an immediate merge rather than use the acceleration lane and have to find a gap in the already merged lane A and lane 1. The exception was study No. 2, in which 53 percent of the ramp vehicles used lane B. (The difference here might have been caused by a bothersome sun with which the ramp drivers had to contend. At least that is the only reason that can be hypothesized other than the vagaries of traffic.)

All in all, this interchange operated smoothly with going-away free-flow volumes averaging 1,200 to 1,550 vph/lane. It was a case of a ramp of high-volume design dominating the scene, especially in the first two studies. However, since completion of the four studies still another section of the Long Island Expressway has been completed and freeway volumes now are close to practical capacity range as they approach the ramp nose. As might be expected, operation is now congested throughout the peak hour, with backups accumulating on both ramp and freeway.

Another 2-lane ramp of interest was that connecting North Conduit Avenue westbound to Van Wyck Expressway northbound on Long Island. The acceleration lane is 520 ft long. The "peak merge hour" produced the following volumes: Ramp lane B, 1,170; ramp lane A, 1,336; merge (ramp B + ramp A + lane 1), 2,917; lane 1, 411; lane 2, 739; lane 3, 491; freeway (lane 1 + lane 2 + lane 3), 1,641; total (freeway + ramp), 4,147.

Similar to the Northern State Parkway - Long Island Expressway ramp, ramp lane A carried more vehicles than ramp lane B. There were numerous lane changes by ramp vehicles jockeying for position as they approached the nose prior to merging. The merge volume of 2,917 vph seems high, but it should be noted that 2,506 were on the ramp with only 411 in lane 1. Although no data were recorded, a number of ramp vehicles did cut directly into lane 2 of the Van Wyck Expressway, which begins at Idlewild Airport 2.5 mi south of this location.

Because traffic volumes are likely to remain quite stable at this location in contrast to those along the previously discussed Long Island Expressway, free-flowing traffic can probably be maintained with present geometrics. One more note—although accident statistics have not been studied, personal observation of this ramp's operation (and to some extent the Long Island Expressway ramp too) led to the belief that safety is sacrificed when ramp vehicles are given three choices of action: (1) Use of acceleration lane and merge into lane 1; (2) Merge directly into lane 1; and (3) Merge directly into lane 2. The last named action caused most of the near misses observed. Eliminating this type of merge would not be easy, as it is more a result of unusual freeway ramp volume distributions than of geometrics or signing.

A 2-lane diamond ramp studied in New Jersey near the Lincoln Tunnel was the Pleasant Avenue on-ramp to New Jersey Route 3 westbound. The 15-min free-flow volume expanded to 1 hour was: Ramp lane B, 452; ramp lane A, 584; merge (ramp B + ramp A + lane 1), 1,524 (28.6% c.v.); lane 1, 488; lane 2, 1,068; lane 3, 1,028; freeway (lane 1 + lane 2 + lane 3), 2,584; total (ramp + freeway), 3,620 (20.2% c. v.).

Principally because a long upgrade on Route 3 ended only 600 ft upstream from the ramp nose, 80 percent of the commercial vehicles were in lane 1. These slow-moving trucks created a number of large gaps, so that once again ramp vehicles favored lane A (N.Y. P.A. - 1 Study in Fig. 20). Ramp lane B vehicles were hampered by a short acceleration lane (180 ft) and by lane A vehicles that were accelerating after direct entry onto lane 1. Observers commented that as many as 20 percent of the merged ramp vehicles moved over into lane 2 within 200 ft of the nose.

The conclusions that can be drawn from the 2-lane on-ramps studied are as follows:

1. Except when the beginning of the freeway is and will remain a short distance upstream, an extra through lane should be added to the freeway.
2. Downstream (after merge) going-away averages exceeding 1,500 vph/lane will usually result in congestive operation. The most probable reason for this is the difficulty in achieving equitable volume distribution among the freeway lanes downstream from high-volume 2-lane ramps. It follows that any time freeway lane averages approaching 1,000 vph/lane upstream from the ramp are expected, an extra through lane should be added to the freeway at 2-lane on-ramp connections. This is especially so for 4- or 6-lane divided freeways.
3. Addition of an extra through freeway lane would help eliminate direct merging into lane 2 by ramp vehicles and increase overall safety, because only one ramp lane would need to merge into lane 1.

OFF-RAMPS

One of the main objectives of the Freeway Ramp Capacity Study is to determine formulas for computing the capacity of the diverging movement from the freeway to the ramp. There is also a need for determining the relative strength of the roles played by such geometric and traffic characteristics as angle of divergence, sight distance, length and shape of deceleration lane, percentage of commercial vehicles, and lane volume distributions. Although work on these objectives is under way, it has not progressed to the point where results can be reported. The observations presented herein are therefore mostly general impressions developed from a review of the off-ramp data submitted.

The capacity problems found at exit ramps are quite dissimilar from those experienced at on-ramps. Whereas the on-ramp driver has the very real task of choosing a gap and merging into it, no such complicated maneuver is necessary at exit ramps. It is rather disconcerting, therefore, that some off-ramps operate in an unsatisfactory manner.

At off-ramps there are three possible capacity limitations, as follows (circled numbers 4, 5, and 6, Fig. 1):

4. The diverging movement from the freeway to the ramp.
5. The ramp proper.
6. The ramp terminal connection to the street system.

The capacity of the diverging movement from the freeway to the ramp has the greatest effect on the through freeway lanes. Reasons for unsatisfactory diverging maneuvers are sometimes difficult to pin down because the origin of the trouble may be some distance upstream or downstream from the ramp.

Some of the causes of diverging difficulties are as follows:

1. Poor signing and/or sight distance, causing abrupt maneuvers or speed changes close to the exit ramp nose.
2. Lack of adequate weaving length on the freeway upstream from the ramp exit, causing excessive lane changing near the nose. Even though advance overhead signing is present, television surveillance lane-changing studies in Detroit disclosed that the

Figure 21. Crowding in at exit nose, illustrating failure to use auxiliary lane to best advantage.

greatest amount of lane-changing took place just upstream of the off-ramp nose. To a certain extent this prevents maintenance of the uniform speed necessary for smooth operation of exiting traffic.

3. The occasional poor usage of auxiliary and deceleration lanes. Cutting-in at the nose of cloverleaf inner loop off-ramps was a problem at several locations, even though an auxiliary lane was available for use by exiting drivers (Fig. 21).

4. Poor operating characteristics of the ramp proper, causing speed reduction to extend back onto the freeway exit lane.

The concept of a capacity of the ramp proper (No. 5, Fig. 1) is really no different than the capacity of the ramp proper for an on-ramp. It is the physical ability of the ramp to handle a continuous supply of vehicles, assuming that there are no limitations at the ramp terminals. It would seem, therefore, to be dependent on such physical characteristics as radius, width, superelevation, and riding surface. Because this ultimate capacity is so seldom reached in practice, this seems to be an area where controlled laboratory experiments, or perhaps simulation, are needed. Of course, if the ramp is tangent, the ramp proper capacity should be the same as conventional freeway lane capacity for the given speed.

The last mentioned capacity limitation (the ramp's connection to the street, frontage road, or interchanging highway system) is a subject in itself. Although some backup along the ramp can be tolerated, the situation reaches serious proportions when the freeway lane 1 is encroached upon. If the exit ramp's terminal is a merging operation into another freeway or expressway (No. 6, Fig. 1), there are not apt to be serious backups unless the traffic is so heavy on the other facility that the ramp vehicles cannot merge without delay. If the ramp is two lanes wide for storage purposes, backups can usually be confined to the ramp. From an operational standpoint, especially at the divergence from the freeway, a 2-lane ramp may be less desirable than a 1-lane ramp. Diamond ramps are almost always 1-lane. It is usually at diamond interchanges controlled by traffic signals that backups become serious enough to extend back onto the freeway. Occasionally, backups occur at diamond off-ramps controlled by stop signs, usually because of difficulties encountered by left-turning vehicles.

Freeway Volume Distribution at Off-Ramps

Freeway volume distributions at off-ramps for 4-, 6-, and 8-lane freeways are given by lanes in Figures 22, 23, and 24. These distributions were taken just downstream from the ramp nose after the ramp traffic had diverged from the freeway stream.

The curves for 6-lane freeways at off-ramp locations (Fig. 23) are derived from 6-lane freeway volume distributions at all types of off-ramps. This figure also gives the volume distributions where an auxiliary lane is present between the off-ramp and the adjacent upstream on-ramp. The auxiliary lane evidently opens up lane 1, because

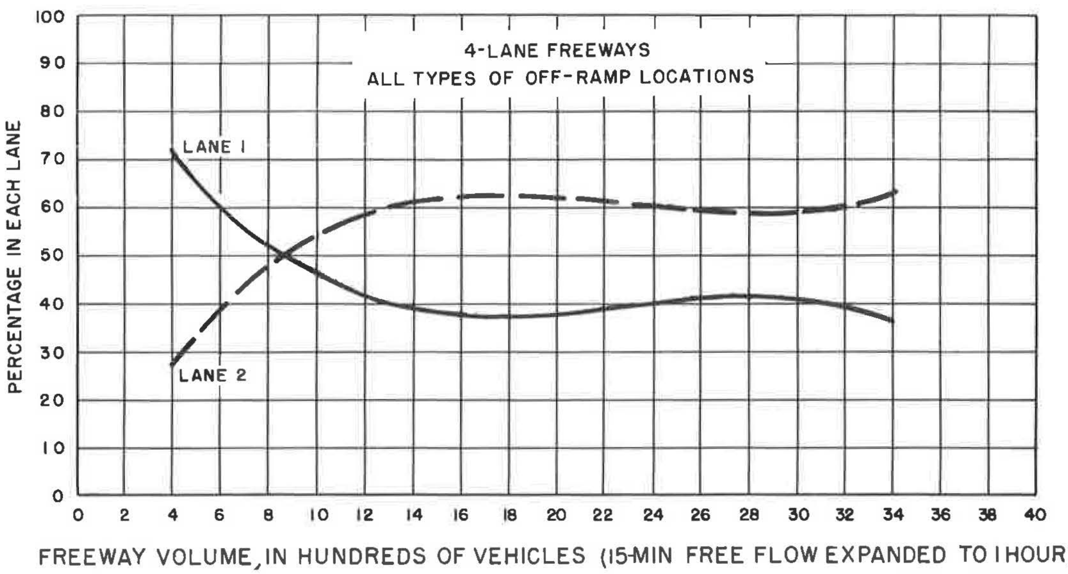


Figure 22. Volume distribution on 4-lane freeways downstream from off-ramps.

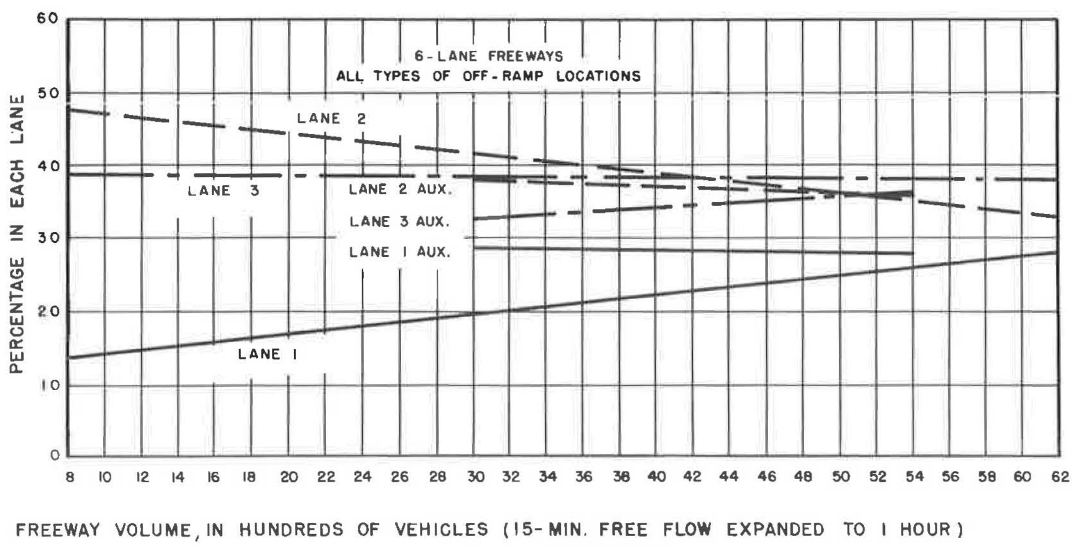


Figure 23. Volume distribution on 6-lane freeways downstream from off-ramps (with and without auxiliary lane upstream).

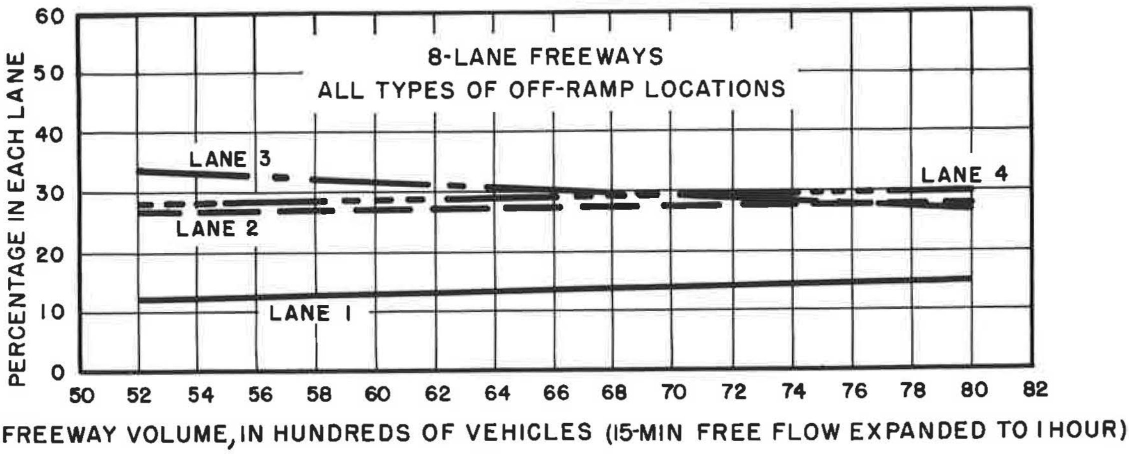


Figure 24. Volume distribution on 8-lane freeways downstream from off-ramps.

where such a lane exists percentages in lane 1 downstream from the off-ramp are 2 to 9 points higher within the volume range shown by the curves. The increased use of lane 1 could also be partially caused by the relative nearness of the adjacent upstream on-ramp.

As stated previously, the distributions shown in Figures 22, 23, and 24 are for the freeway downstream from the ramp nose. This explains the low percentage of freeway traffic in lane 1. If the lane percentages were shown as taken upstream from the ramp (before movement to the deceleration lane or ramp by drivers intending to exit), lane 1 would usually carry the highest percentage of the freeway volume at lane volumes below practical capacity. The bunching of vehicles in lane 1 upstream from exit ramps is not a desirable characteristic. The seeming inability of the freeway traffic to evenly distribute among lanes upstream from off-ramps at volumes below practical capacity is being investigated as a primary contributor to congestion at exit ramps. An exit ramp which requires a considerable reduction in speed at the diverge from lane 1 is especially apt to cause erratic operation.

At volumes above practical capacity, a more ideal utilization of the freeway lanes upstream from exit ramps is accomplished. Lane 1 will often carry the lowest percentage; this is good, because it is the "action lane" subject to the most disturbance from the ramp vehicles.

Vehicle Storage at Off-Ramps

As previously mentioned, one of the problems frequently confronting traffic engineers is that of alleviating major backups on diamond off-ramps. Sometimes the remedy is merely to provide sufficient turning lanes to help in getting the traffic off the ramp and onto the surrounding street system. In other cases, heavy traffic on the surrounding street system complicates the problem so that a solution must be found to absorb the ramp traffic into the local traffic without unduly disrupting overall flow. A good signal system is important, but even at its optimum setting it is not always possible to keep ahead of the high exit volumes encountered over short periods. There must be room to store these vehicles in the interim.

Tying a diamond ramp to a parallel frontage road, either continuous or non-continous, appears advantageous when the frontage road is not heavily used by through vehicles. In such cases, where adequate weaving distance is available between the ramp-frontage road junction and the cross street, maneuvering of the ramp vehicles will be facilitated and serious backups will be eliminated. On the other hand, the connection of diamond ramps to heavily used frontage roads at points only a few hundred feet from the cross street was a cause of trouble at several study locations; not only did the frontage road vehicles monopolize the green signal time, but they hindered ramp vehicles attempting to obtain access to the desired frontage road lane. Ramp drivers wishing to turn right at the cross street were especially hampered.

The operational problems and capacity limitations at diamond interchanges are important enough to warrant research projects in their own right. Highway Research Board Bull. 291 contains a recent research study (4) along these lines.

ACKNOWLEDGMENT

The nomographs presented in this text were derived by Mrs. Beverly Norris, Highway Research Engineer, Traffic Research Branch, and Steiner Silence, Traffic Engineer, Region 4, U.S. Bureau of Public Roads.

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Appendix A

REGRESSION ANALYSIS

Discussion of Variables

The problem to be solved is the determination of free-flow merge volumes for various freeway and ramp volumes and for various geometric configurations. A regression analysis for one-lane right-hand on-ramps was run using 73 observations (Table 4) from 50 locations. Lack of sufficient two-lane on-ramp studies prevented any reliable regression analysis in that category.

All types of right-hand on-ramps were included in the one-lane on-ramp regression analysis. These included 16 diamonds, 12 cloverleaf outer connections, 6 cloverleaf inner loops, 8 slips, 3 directs, 4 semidirects, and 1 trumpet outer connection. A number of these ramps were studied several times, not only during morning and evening rush hours but also on different days. As discussed later, several of these ramp types were later deleted, finally reducing to 55 observations, from which an additional more limited formula was obtained.

The formulas contain six independent variables expressing traffic characteristics, geometrics, and community size. The glossary should be consulted for detailed definitions of these variables. The dependent variable, free-flow merge volume in vph, is the unknown quantity desired. It should be emphasized that computations are made for merge capacity and not primarily for the number of ramp vehicles which can enter onto the freeway without congestion. Once the free-flow merge has been computed, and it should be kept clearly in mind that this is not a constant for all combinations of traffic, the only remaining step necessary is to subtract the lane 1 volume from the merge volume to determine the ramp's capacity at the given freeway volume.

Some explanation of the independent variables is in order. One might wonder why X_1, the percent freeway utilization, is used as a variable. This is necessary as a reflection of demand. As a "plus" quantity, the higher the freeway utilization, the higher the free-flow merge volume determined by the formula. At very high freeway volumes, say 90 percent freeway utilization, a high merge volume can be expected, although most of these merging vehicles will be in lane 1. The volume which could be accommodated by the ramp would be low in this case.

TABLE 4

FREE-FLOW MERGE REGRESSION ANALYSIS

Item	Variables						
	Y F. F. Merge	X_1 % Fwy. Util.	X_2 % Comm. Veh. in Merge	X_3 Ramp/Merge Ratio	X_4 Angle of Convergence	X_5 Length of Accel. Lane	X_6 Metrop. Area Pop.[a]
73 Observations[b] (Eq. A):							
Type of variable	Dep.	Indep.	Indep.	Indep.	Indep.	Indep.	Indep.
Units	Vph[c]	% × 100	% × 100	% × 100	Degrees	Feet	1,000's
Mean	1569.2	67.9	4.8	38.0	13.9	418.2	2549.7
Std. deviation	287.6	14.9	4.2	15.4	10.4	258.5	1462.2
Net regression coefficient	1.0	+8.5	-16.5	+7.6	-1.0	+0.22	+0.071
Std. error of net regression coefficient	0.0	2.2	5.4	1.9	2.2	0.09	0.019
Partial determination coeff.	1.0	0.19	0.13	0.20	0.00	0.08	0.17
Level of significance	–	0.01	0.01	0.01	–[d]	0.02	0.01
55 Observations[e] (Eq. B):							
Mean	1616.3	69.9	4.8	38.7	12.1	419.5	2768.2
Std. deviation	277.3	15.4	4.5	14.9	9.5	267.8	1451.0
Net regression coefficient	1.0	+10.1	-17.9	+9.5	-4.8	+0.14	+0.068
Std. error of net regression coefficient	0.	2.3	5.7	2.3	2.6	0.10	0.020
Partial determination coeff.	1.0	0.28	0.17	0.26	0.07	0.04	0.19
Level of significance	–	0.01	0.01	0.01	0.10	0.20	0.01

[a]Value used in formula should not exceed 5,000. [b]Const. = 527.7, R^2 = 0.68, std. error of Y = 169.8, std. error/mean = 0.108. [c]15-min f.f. expanded. [d]Not significant. [e]Const. = 441.3, R^2 = 0.71, std. error of Y = 158.9, std. error/mean = 0.098.

X_2, the percent commercial vehicles in the merge (% c.v. in merge), is a "minus" quantity. The simple negative correlation between this variable and the free-flow merge is shown in Figure 25 with cloverleaf inner loops treated as a separate category from other types of ramps.

X_3, the ramp volume/merge volume ratio, is another traffic characteristic of primary importance because it is an indication of ramp demand and availability of gaps in lane 1. This ratio has already been discussed in connection with the proportions of ramp and lane 1 volumes for free-flow merge, as shown in Figures 13 and 14.

The geometric variables used are X_4, angle of convergence, and X_5, length of acceleration lane. The former has a minus correlation, as shown in Figure 26. For X_5, the free-flow merge volume has a positive correlation with the length of acceleration lane (Fig. 27). Although the regression analysis is concerned exclusively with one-lane on-ramps, several 2-lane ramps were included in the data determining the curve in Figure 27. Accordingly, the maximum free-flow merge of 2,100 vph is not outside capability, because the 2-lane ramps had long acceleration lanes. Also, some ramp

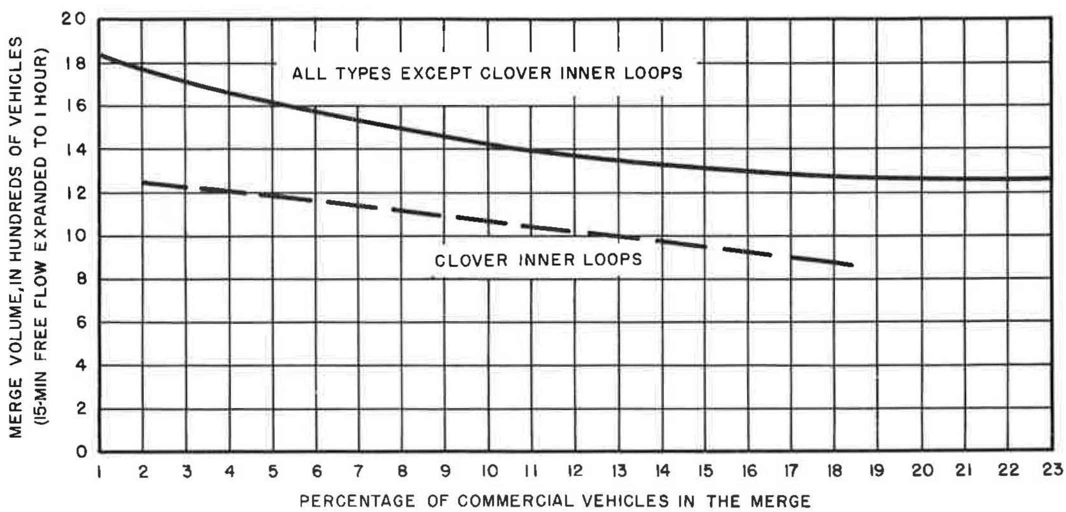


Figure 25. Free-flow merge volumes vs percent commercial vehicles in merge.

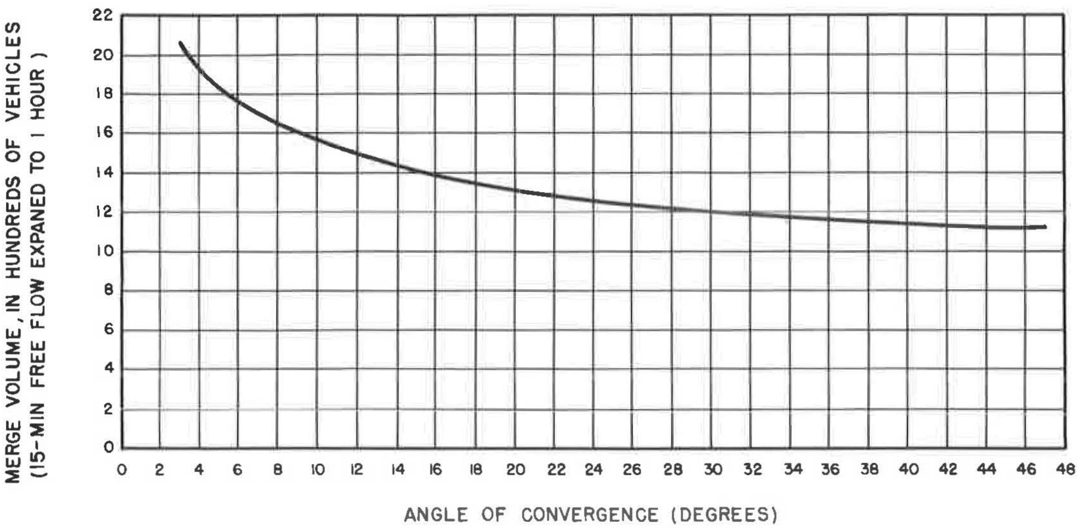


Figure 26. Free-flow merge vs angle of convergence of on-ramp with freeway.

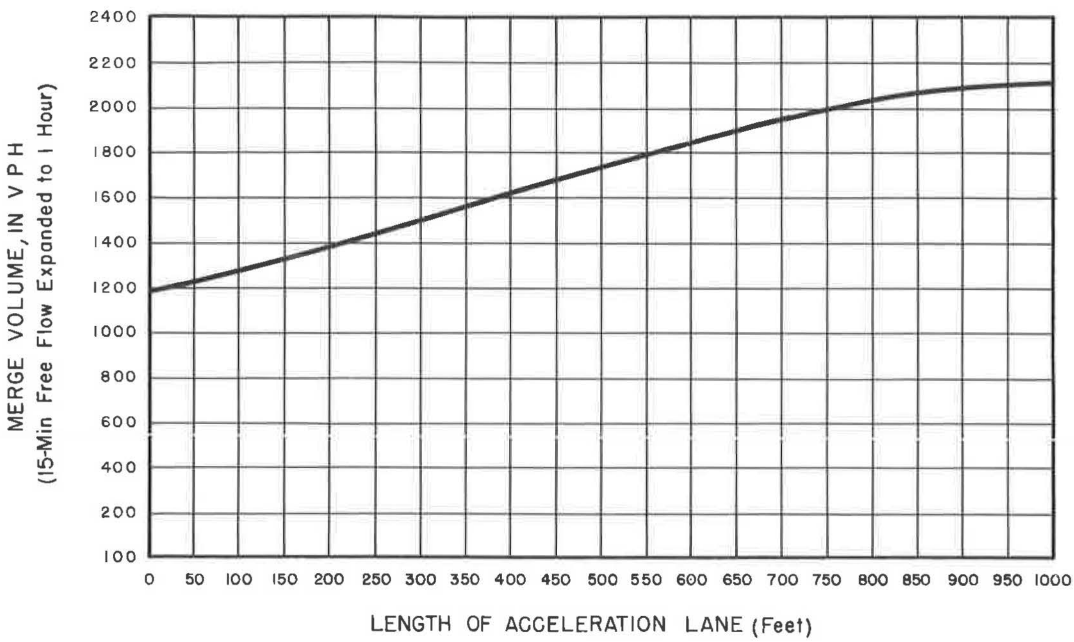


Figure 27. Free-flow merge volume related to length of acceleration lane for 1-lane and 2-lane on-ramps, combined.

vehicles actually merged directly into lane 2, because the freeway volumes were in several cases quite low.

These two geometric variables, X_4, angle of convergence, and X_5, length of acceleration lane, admittedly do not cover all the geometric features of ramp-freeway connections. To name a few, consideration could be made of the width of ramp, shoulders, grades, and particularly sight distances. None of these had much variation in the 73 observations. The effect of a steep sustained uphill grade can be substantial where commercial vehicles are involved, but the studies submitted for this project contained few freeway grades exceeding 2 percent. Grades were considered in the analysis, but no significant relationship was found, although it is possible that this could have been a significant variable if there had been more variation in grades among the study locations. An upgrade ramp with poor sight distance certainly does not lend itself to free-flow operation, but these are not common on modern freeways and none were included in the study.

Sight distance at volume levels below practical capacity is undoubtedly important even when the ramp driver is still back 400 ft from the ramp nose. However, when volumes increase to the levels usually found during peak hours, the on-ramp driver faces a later decision in sizing up lane 1 traffic, now heavier, but traveling at a more uniform and usually slower speed than at volumes below practical capacity. The driver necessarily has to limit his vision to those lane 1 vehicles which are within several hundred feet of the ramp nose. At higher ramp volumes, the ramp driver must also be more concerned with fellow ramp drivers, especially those immediately ahead of him. In essence, sight distance for the last 200 ft traversed along the ramp is all that really matters at high-volume levels. Sight distances at practically all the study sites were adequate in this sense—the driver could see as much as he needed to see as he moved along the ramp just before entering the acceleration lane or lane 1 (if an acceleration lane was unavailable). Sight distance, as such, is really accounted for in the two geometric variables. A narrow angle of convergence usually helps simplify the task of sizing up gaps in lane 1, whereas the presence of an acceleration lane assures the driver of the necessary time to see and choose a gap for merging.

The use of X_6, metropolitan area population expressed in thousands, might be questioned because it is outside the scope of geometrics and traffic characteristics usually associated with capacity calculations. Past experience in intersection capacity pointed toward its inclusion and the results of the analysis, where it was given a trial deletion, confirmed the need for this variable.

Some other possible variables which were tried and found insignificant, within the limits of the study, were the number of freeway lanes in one direction and the percentage of commercial vehicles in the freeway lanes other than lane 1.

Effect of Variables

Table 4 presents a more detailed look at the variables used and their relative significance. The R^2, the coefficient of determination or measure of explained multiple correlation for the entire formula, of 0.68 was slightly exceeded several times in the computer runs for 73 observations. However, the formula chosen was the best from the standpoint of practical application and its standard error of estimate of 169.8 vph was only a few vehicles per hour higher than the lowest standard error of estimate obtained from the various runs. The partial determination coefficients (r^2) for each of the variables are also given.

Using the student's "t" test, all of the variables except X_4, angle of convergence, were found significant at the 0.02 level. It was decided to retain X_4 because, from practical experience, it seemed that this variable should be important. As is discussed later, it is significant at the 0.10 level after deletion of 18 observations comprising the cloverleaf inner loops and slip ramps. Accordingly, it is retained as a variable in each of the formulas.

One of the most significant variables is X_6, the metropolitan area population. Undoubtedly, a major reason for this importance is less fluctuation in demand in the larger cities, enabling a more stable flow at given volume levels. Figure 2, which shows the percent of the peak hour of the peak 5-min period within the peak hour, indicates less pronounced short-period peaking in the large cities.

Too, there are other factors aside from less fluctuation in demand in the large cities. One of the these is driver experience, a necessary part of smooth high-volume operation. Drivers in the large metropolitan areas have had much more freeway experience in the past decade than their counterparts in the smaller cities such as Jacksonville, Fla., or Columbus, Ohio. On a daily driving basis, the larger cities with their more extensive and often better designed freeway networks provide more merging situations. In a city like Chicago, the driver is expected to merge into the traffic stream without stopping, and he does. In a smaller city this compulsion is not so great. A stopped car often blocks those following and the low entry speed of the resultant queue tends to cause congestive operation at lower volume levels. Time and again, observers in the smaller cities commented on the unnecessary stops by ramp vehicles. As the Interstate urban sections are completed, driver skills should develop, but even after the completion of the Interstate system, the large cities should still offer more daily freeway driving and a more uniform flow of traffic at given volume levels. Aside from experience, but probably closely allied to it, is the aggressiveness which characterizes big city drivers. This aggressiveness is part of the generally faster pace of life in the large cities and is certainly not a deficiency in courtesy. In fact, there is probably more beneficial give-and-take driving in the larger cities.

However, in using the formulas, a metropolitan area population exceeding 5,000,000 would add an inordinate amount of vehicles to the answer. This would indicate more "merging ability" superiority than actually exists for the few cities with very large metropolitan area populations, such as New York. For this reason, the largest X_6 value used should not exceed 5,000. This figure should be used for New York City, Los Angeles, and Chicago, the only metropolitan areas exceeding 5,000,000 population.

The designer might be in a quandary as to the population figure which should be assigned to an interchange located outside city limits; for example, one located in an unincorporated area of New Jersey near New York City. In such a case, judgment would have to be used in determining the type, the origin, and the destination of drivers using the interchange. For instance, if the interchange was near Ridgefield Park, primarily serving commuters of Passaic and Bergen counties enroute to and from Hudson County, N.J., and New York County (Manhattan), the designer could use the combined populations of these counties. For Bergen, Passaic, Hudson, and New York Counties, this total would be 3,495,888 from the 1960 census. Accordingly, 3,496 would be entered as the X_7 value.

Deletion of Observations

To obtain a more homogeneous sample, the ramps of very short length and the ramps of sharp curvature or short radius near the nose were deleted from the 73 observations. The ramps having these features were the slip and cloverleaf inner loop ramps. Deletion of these ramps left a remainder of 55 observations comprised of 27 diamonds, 17 cloverleaf outer connections, 6 semidirects, 4 directs, and 1 trumpet outer connection. These observations were from 37 locations.

The formula obtained from the 55 observations had an R^2 of 0.71 and a standard error of estimate of 158.9 vph. The mean input value of the free-flow merge was 1,616 vph, or 47 vph more than the mean for the 73 observations. Because the deleted slip-ramps and cloverleaf inner loop ramps were generally of poorer geometrics, this difference is as expected.

The student's "t" tests for significance of variables disclosed significance at the 0.01 level for all the variables except X_4 and X_5. These two variables, the angle of convergence and the length of acceleration lane, were significant at the 0.10 and 0.20 levels, respectively. X_4, the angle of convergence, now has more importance than formerly with a coefficient of -5.0. For the 55 observations, the average angle of convergence was 8° for diamond ramps and 22° for cloverleaf outer connections. If only the angle of convergence is considered, there would be an average difference, attributable to the X_4 coefficient of -5.0, of 70 vph in the free-flow merge volume for the diamond and cloverleaf outer connection ramps included in the analysis. The foregoing is stated only to give a general indication of the contribution of X_4 to the formula. As in any multiple regression formula, final conclusions should not be based on an interpretation of the effect of any single variable. Primary importance should be attached to the computed free-flow merge which takes into account all the variables in the formula.

Appendix B

LANE 1 VOLUME EQUATIONS

In freeway driving, as well as in any other type of driving, drivers strive to obtain a certain degree of comfort or freedom. The hoped-for optimization of this freedom is attempted by the individual driver by choosing the traffic lane which promises the least conflict compatible with his intended destination. The reactions of each driver and the distribution of the freeway traffic in general are primarily dependent on the freeway volume level. It is unlikely that a six-lane freeway would carry 1,900 vph in lane 3 while carrying only 1,000 vph in lane 2, although such a condition is entirely possible without a breakdown in either lane. Most certainly, some of the drivers in lane 3 would move to lane 2 to achieve more comfortable headways. In the same sense, lane 1 volume is also primarily dependent on the freeway volume; this is especially the case just upstream from an on-ramp connection. Downstream from the ramp, the volume in lane 1 would, to a considerable extent, be dependent on the newly merged ramp volume.

The use of curves for lane volume percentages, based on varying freeway volumes, is one method of determining the lane 1 volume used in the free-flow merge calculations. Although the curves are based on freeway volumes, they represent averages of all the other factors which influence the use of lane 1. Some of these factors could be signing, adjacent ramp volume and distance, study ramp pressure on lane 1, commercial vehicles (especially where sustained upgrades are encountered), locality of the interchange, trip lengths, and the spacing of interchanges. These determinants, although not as important as the total freeway volume, nevertheless have some effect on the freeway volume distribution.

Accordingly, another method, applicable within the limits of the available data, is the use of lane 1 volume equations developed by multiple regression analysis. These equations contain not only the freeway volume as a variable, but also the ramp volume and adjacent ramp action. Use of these additional factors makes possible an increase in the accuracy of lane 1 volume calculations, especially at freeway volume levels below practical capacity. Nomographs (Figs. 10, 11, and 12) representing Eqs. No. 1, 4, and 5 are available for graphic solution of problems.

The available data were sufficient for only the more common freeway layouts, as shown by the sketches in Table 5 and the adjacent ramp distance and volume ranges for each equation as given in Table 3.

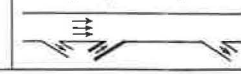
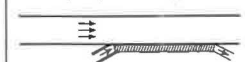
Equation No. 1

Eq. No. 1 (Table 5) is used for determining the lane 1 volume upstream from the on-ramp nose for 6-lane freeways where there are adjacent upstream and downstream off-ramps. There is no auxiliary lane between the on-ramp and the adjacent downstream off-ramp.

The data used to develop this equation consisted of 325 free-moving 5-min traffic counts from the eastern end of the Edsel Ford Expressway in Detroit, from the Gulf Freeway in Houston, and from the Cross Island Parkway on Long Island. The Detroit data consisted of 266 5-min traffic counts from nine on-ramp locations and their adjacent ramps which were counted simultaneously. These on-ramps consisted of 6 diamonds, 1 direct connection, and 2 cloverleaf outer connections. The cloverleaf outer connections were from a partial cloverleaf interchange and, although there were adjacent upstream inner loop off-ramps, there were no upstream inner loop on-ramps to cause weaving at the interchange. The 6-mile freeway section in Detroit was a smoothly operating freeway at high volumes and quite typical of a modern radial depressed facility. It was opened to traffic in 1958-9. The Houston ramps were both slip types, which provided 42 5-min traffic counts. The Long Island ramp was a cloverleaf outer connection with no upstream inner loop on-ramp. The 5-min counts used in the regression analysis were from periods when the traffic was moving steadily without stop-and-go operation. Naturally enough, at volumes near possible capacity the speeds could be in the 25- to 30-mph range. It was decided to include some vol-

TABLE 5

FORMULAS FOR LANE 1 VOLUME

6-LANE FREEWAYS	VARIABLES: DEPENDENT — Y Volume in Lane 1 v.p.h.	INDEPENDENT — X_1 Freeway Volume v.p.h.	X_2 Ramp Volume v.p.h.	X_3 Distance in Feet to Adjacent Upstream Off-Ramp	X_4 Vol. of Adjacent Upstream Off-Ramp v.p.h.	X_5 Distance in Feet to Adjacent Downstream Off-Ramp	X_6 Vol. of Adjacent Downstream Off-Ramp (v.p.h.)	$\frac{X_6}{X_5}$	Const	R^2	Std. Error of Y v.p.h.	Std. Error Mean
With Adjacent Off-Ramps									-121	.80	140	.134
Mean	1041	4327	594	1459	465	2482	449	--				
Standard Deviation	312	943	349	558	251	1404	260	--				
Range of Use of Variable in Equation		2400 - 6200	100 - 1700	900 - 2600	50 - 1100	900 - 5700	50 - 1300	--				
Net Regression Coefficient		+.244			-.085			+640				
Std. Error of Net Regr. Coefficient		.008			.035			50				
Partial Determination Coefficient		.72			.02			.34				
Level of Significance		.01			.01			.01				
Equation No. 1	Vol. in Lane 1 (v.p.r.) = -121 + .244 (fwy. vol.) - .085 (vol. of upst. off-ramp) + 640 (Vol. of Downstream off-ramp)/(Dist. to Downstream off-ramp)											
With Adjacent Upstream Off-Ramp and Auxiliary Lane Downstream									+62	.77	139	.127
Mean	1099	3876	638	1513	407	775*	496	--				
Standard Deviation	287	1050	363	703	232	160	187	--				
Range of Use of Variable in Equation		1900 - 6200	150 - 1900	450 - 2150	50 - 1000	550 - 950	50 - 1000	--				
Net Regression Coefficient		+.232	-.072	-	-	-.041	+.432	--				
Std. Error of Net Regr. Coefficient		.015	.047	-	-	.108	.068	--				
Partial Determination Coefficient		.67	.02	-	-	.00	.25	--				
Level of Significance		.01	.01	-	-	N.S.**	.01	--				
Equation No. 2	Vol. in Lane 1 (v.p.h.) = 62 + .232 (fwy.vol.) - .072 (ramp vol.) - .041 (length aux. lane) + .432 (vol. of downstream off-ramp)											
Auxiliary Lane Downstream No Significant Upstream Adjacent Ramp									-160	.80	174	.154
Mean	1130	4062	678	Not	Not	756*	492	--				
Standard Deviation	387	1138	363			149	205	--				
Range of Use of Variable in Equation		1900 - 6200	50 - 1900			550 - 950	50 - 1000	--				
Net Regression Coefficient		+.273	-.195	Considered	Considered	-	+.633	--				
Std. Error of Net Regr. Coefficient		.014	.042			-	.070	--				
Partial Determination Coefficient		.71	.12			-	.35	--				
Level of Significance		.01	.01			-	.01	--				
Equation No. 3	Vol. in Lane 1 (v.p.h.) =-160 + .273 (fwy. vol.) - .195 (ramp vol.) + .635 (vol. of downstream off-ramp)											
Cloverleaf Inner Loop with Auxiliary Lane to Inner Loop Off-Ramp									-87	.64	178	.203
Mean	878	3792	810	Not	Not	-	447	--				
Standard Deviation	293	790	383			-	282	--				
Range of Use of Variable in Equation		2?0 - 5600	200 - 1500			450 - 850	150 - 1500	--				
Net Regression Coefficient		+.225	-.140	Considered	Considered	-	+.500	--				
Std. Error of Net Regr. Coefficient		.019	.040			-	.054	--				
Partial Determination Coefficient		.51	.08			-	.39	--				
Level of Significance		.01	.01			-	.01	--				
Equation No. 4	Vol. in Lane 1 (v.p.h.) = -87 + .225 (fwy. vol.) - .140 (ramp vol.) + .500 (vol. of downstream off-ramp)											
4-LANE FREEWAYS												
With Adjacent Off-Ramps									+55	.92	76	.086
Mean	874	2304	436	716	166	2582	259	--				
Standard Deviation	270	672	239	462	140	1142	167	--				
Range of Use of Variable in Equation		1100 - 3700	50 - 1000	100 - 1450	50 - 600	1000 - 5000	50 - 750	--				
Net Regression Coefficient		+.363	-.184	-	-	+.022	+.030	--				
Std. Error of Net Regr. Coefficient		.009	.028	-	-	.007	.044	--				
Partial Determination Coefficient		.90	.20	-	-	.05	.00	--				
Level of Significance		.01	.01	-	-	.01	N.S.**	--				
Equation No. 5	Vol. in Lane 1 (v.p.h.) = 55 + .363 (fwy. vol.) - .184 (ramp vol.) + .022 (dist. to downstream off-ramp) + .030 (vol. of downstream off-ramp)											

*Same as length of auxiliary lane.
**Not Significant.

umes as high as possible capacity in the analysis as long as the freeway traffic was moving in a steady manner without stop-and-go.

Except for the combination of the downstream off-ramp volume and distance in the last term, each variable was assumed linear in the multiple regression analysis. Although correlation of the lane 1 volume with the total freeway volume is slightly curvilinear, the relationship is not pronounced enough to warrant transformation of the variable. The R^2 (coefficient of determination or explained variance) of 0.80 and the standard error of 140 vph are indications of a good fit.

Several variables other than those contained in the formula were tried. One was the percentage of commercial vehicles in the merge, which was found insignificant within the limits of the study data. However, commercial vehicles can dominate lane 1 use on sustained upgrades. The angle of convergence of the ramp with the freeway and the length of acceleration lane were also tried as variables, but it was decided that there were insufficient locations to give a valid range of input values. The other variables (not used) which affect lane 1 volume are evidently not too important individually. Some of them, such as trip length, would be difficult to measure or to apply in a formula.

A final word of caution—Eq. No. 1 should not be used for cloverleaf inner loop on-ramps where there is an adjacent downstream cloverleaf inner loop off-ramp. The close-in weaving between the inner loop ramps could change the freeway volume distribution so as to invalidate the equation. As shown in Table 3, the adjacent downstream off-ramp should be a minimum of 900 ft away. This limitation automatically rules out most conventional cloverleaf inner loop connections. Eq. No. 4 can be used at cloverleaf inner loops having auxiliary lanes.

Equation No. 2

As shown in Table 5, Eq. No. 2 is used for determining lane 1 volume upstream from the on-ramp nose where there is an adjacent upstream off-ramp and a downstream off-ramp less than 1,000 ft away connected to the study ramp by an auxiliary lane.

The data used consisted of 128 observations, of which 63 were 5-min periods and 65 were 1-min periods. The 63 5-min periods were from two locations—a cloverleaf inner loop connection, studied both morning and evening peak hours, on the Valley Highway in Denver, and a slip ramp connection on the Gulf Freeway in Houston. The 65 1-min periods were from a diamond connection on the Edsel Ford Expressway in Detroit.

Table 5 indicates that the distance away and volume of the adjacent upstream off-ramp are not contained in the equation. However, as explained in the main text, this upstream off-ramp should be present and within the limits set up in Table 3 before Eq. No. 2 becomes valid. The length of the auxiliary lane, X_5, although not found significant is nevertheless contained in the equation. This logically could be a significant variable but there was not enough variation in the auxiliary lane lengths for the study locations used to prove it so here.

It is rather interesting to note the strong effect (high coefficient) of the downstream off-ramp volume as contrasted with the same variable in Eq. No. 1. The increased effect is apparently because the prospective off-ramp traffic loads up lane 1 just upstream from the on-ramp in anticipation of moving over into the rather short (550 to 950 ft) auxiliary lane. The main on-ramp connection now exerts less pressure on the lane 1 volume than formerly.

Equation No. 3

This equation is used for determining lane 1 volume upstream from the on-ramp nose for 6-lane freeways when the on-ramp under consideration is connected to an adjacent off-ramp less than 1,000 ft downstream by an auxiliary lane. Also, there is either no nearby upstream ramp, or if so its volume or effect on through traffic is negligible.

This equation extends the data contained in Eq. No. 2 to include 35 additional 5-min periods, making a new total of 163 observations. The 35 additional observations come from a cloverleaf inner loop on Long Island and a diamond ramp in Los Angeles. In the case of the Long Island ramp there is no adjacent upstream outer connection off-ramp.

Again, the equation is restricted to locations where the auxiliary lane to the downstream off-ramp is less than 1, 000 ft in length.

Equation No. 4

Eq. No. 4, intended for use in determining lane 1 volume on 6-lane freeways just upstream from inner loop on-ramps at cloverleaf interchanges, includes as independent variables the freeway volume upstream from the ramp nose, the inner loop on-ramp volume, and the downstream off-ramp volume. The inner loops should be connected by an auxiliary lane in the usual range of length (450 to 850 ft). The basic data come from loops without outer connections and from loops with outer connections (conventional cloverleafs). Because of lack of volume data, the outer connection operation, when present, was not included as variables in the equation. Undoubtedly, an upstream outer connection off-ramp, if present, would tend to reduce the lane 1 volume at the inner loop nose. This omission, together with the more pronounced natural variation of operation at cloverleafs, helps to account for the higher standard error of estimate at cloverleafs, helps to account for the higher standard error of estimate (178 vph) and lower R^2 (0. 64) of this equation as compared with the other equations. This is not to distract from its superiority over using curves based on freeway volume only. The ramp volumes on inner loops have a great effect on the lane 1 volume.

The basic data, consisting of 136 5-min observations, comes from seven locations on Long Island, along the Edens Expressway in Chicago, and from the Whipple Avenue interchange of the Bayshore Freeway near San Francisco.

Equation No. 5

This equation is used for determining lane 1 volume upstream from an on-ramp nose for 4-lane freeways where there are adjacent upstream and downstream off-ramps. There is no auxiliary lane between the on-ramp and the adjacent downstream off-ramp. Thus, the situation is the same as for Eq. No. 1 except that the freeway is 4 lanes instead of 6 lanes.

The data used consisted of 187 5-min traffic counts from seven locations in Denver, St. Louis, San Antonio, and San Jose. These ramps included 4 cloverleaf outer connections, 1 diamond, 1 semidirect connection, and 1 partial cloverleaf inner loop on-ramp with no following inner loop off-ramp. The R^2 of 0. 92 and standard error of 76 vph of the equation are excellent, especially considering that only four independent variables are used.

Although the distance to and volume of the adjacent upstream off-ramp are not contained in the equation, such a ramp must be present within the ranges shown in Table 3 before the equation may be applied. Herein lies the biggest weakness of this equation, because the adjacent upstream off-ramp volume could possibly exceed the 50- to 600 vph range specified in Table 3. Volumes and distances outside the specified ranges could have an effect on lane 1 volumes which the equation could not accurately fit. Figure 3 is applicable to situations which fall outside the specified ranges.

Figure 3 should also be used whenever the connection being considered is a cloverleaf inner loop on-ramp, because Eq. No. 4 does not apply to this type of layout. The weaving to the downstream off-ramp causes a different lane 1 volume curve, as shown in Figure 3. However, if the ramp is a cloverleaf outer connection or a cloverleaf inner loop (at a partial cloverleaf interchange) with no adjacent downstream inner loop off-ramp, the equation is applicable.

Appendix C

OPERATION OF A DIAMOND ON-RAMP

Most of the interchanges on the Edsel Ford Expressway in Detroit are of the diamond type. Usually the ramps connect to a service road 300 to 600 ft from the cross street. The eastern end of the Expressway, which was opened to traffic in 1958-9, has some very efficient ramp-freeway connections. The one presented here is the Mt. Elliot on-ramp to the Expressway westbound (inbound in the morning). The study period was from 6:00 to 8:10 a.m. The first hour of operation was free-flowing at speeds of 45 to 55 mph. The interval from 7:03 to 7:09 was a period of gradually decreasing speeds as the freeway became saturated at the merging area. From 7:09 to the end of the study at 8:10, speeds were erratic and there was some stop-and-go traffic. Only a few times, for short intervals only, did speeds get up to 30 mph.

The period from 6:30 to 7:00 was a period of good demand and high volumes at speeds of 45 to 55 mph. Most of the elements of optimum operation are present at this location—level, tangent expressway, narrow angle of convergence (6°), and good sight distance. The ramp had a 500-ft tapered acceleration lane, which is close to the average length of acceleration lane for the various studies submitted nationally. The half-hour period expanded to one hour had the following volumes: ramp, 478 vph, 9.6 percent trucks; lane 1, 1,348 vph, 6.4 percent trucks; merge, 1,826 vph (ramp + lane 1); lane 2, 1,986 vph; lane 3, 2,150 vph; freeway, 5,484 vph (lane 1 + lane 2 + lane 3); total, 5,962 vph, 2.3 percent trucks (freeway + ramp); going-away average per lane = 1,987 vph.

During the half-hour period, 378 vehicles entered the service road at the cross street with 239 of these going down the ramp and 139 staying on the service road past the ramp. In other words, approximately 63 percent of the vehicles on the service road used the ramp during free-flow freeway operation.

Nearby ramps counted at the same time were the upstream off-ramp (1,580 ft away) to Mt. Elliot with an expanded volume of 320 vph and the downstream off-ramp (1,860 ft away) to Chene with an expanded volume of 598 vph. It might be noted that, using Eq. No. 1 the expanded volume calculated for lane 1 would have been 1,396 vph, which is 48 vph more than the 1,348 vph actual count. This is a 3.6 percent error, which in this case would cause a slightly higher computed "expected merge" volume. Weaving was quite noticeable downstream from the study ramp location as vehicles in lanes 2 and 3 moved over to exit at the Chene ramp.

At the main study ramp the flow rate (total freeway stream after merge) for each of the six 5-min periods in the half hour considered was 5,604, 5,340, 5,736, 5,868, 6,276, and 6,984 vph. As can be seen, the traffic buildup was steady to the breakdown point, which occurred 3 min after the 5-min period during which the 6,984-vph rate was recorded. Because the breakdown definitely occurred at the study location, the 2,316-vph/lane average for the last 5-min period may have some significance as representing a maximum 5-min capability downstream from a ramp. Short-period volumes in this range have been obtained at other smoothly operating sections.

As mentioned before, the period from 7:10 to 8:10 was a period of erratic and low-speed traffic flow. The following volumes for this hour illustrate quite well that as long as demand is steady and continuous heavy volumes of traffic can pass a point even under these conditions: ramp, 842 vph, 8.8 percent trucks; lane 1, 1,378 vph, 6.0 percent trucks; merge, 2,220 vph (ramp + lane 1); lane 2, 1,993 vph; lane 3, 2,008 vph; freeway, 5,379 vph (lane 1 + lane 2 + lane 3); total, 6,221 vph, 2.6 percent trucks (freeway + ramp); going-away average per lane, 2,074 vph.

Of the 1,743 vehicles on the service road during this hour period, 842 went down the ramp. This 48 percent use of the ramp, as contrasted with 63 percent use during free flow, suggests that some drivers stayed off the expressway because of its congested state. The option afforded the service road driver is one of the big advantages of connecting diamond ramps to service roads rather than directly to the cross-street.

Finally, mention should be made of some high lane counts per 5-min period. During the free-flow period, lane 2 had 5-min counts of 182 and 198 vehicles and lane 3

Figure 28. Overhead signing for Chalmers exit ramp from Edsel Ford Expressway eastbound in Detroit.

Figure 29. Chalmers exit ramp from Edsel Ford Expressway; volume on expressway upstream from ramp is near possible capacity.

had counts of 192, 185, 186, and 213 vehicles. During the full hour of congested operation, four counts in the 190 to 200 range were recorded for lane 2. Expansion of these 5-min counts to an hourly rate gives volume rates ranging from 2,184 to 2,556 vph/lane.

OPERATION OF A DIAMOND OFF-RAMP

A diamond off-ramp studied in Detroit was the Chalmers off-ramp from the Edsel Ford Expressway eastbound. This ramp is relatively far out on the expressway (6½ mi from the Ford-Lodge interchange) in a residential area. The expressway at the location is level and straight. The ramp exit sign (Fig. 28) could be seen when still 0.4 mi upstream from the nose of the ramp (Fig. 29), though it probably would not ordinarily be seen when that far away by the strangers most in need of it. Although the freeway volumes were near possible capacity when the pictures were taken, the volumes appear to be much lower. A tapered deceleration lane 400 ft long precedes the nose. Practically all the ramp vehicles used the complete deceleration lane, beginning the turn off lane 1 at the beginning of the taper. Speeds in lane 1 were not decreased for this maneuver as far as could be determined (average 45 mph, with lanes 2 and 3 traveling 51 to 57 mph). A number of drivers used turn signal indicators (approximately 40 percent during spot checks). The ramp exits to the service road 980 ft before reaching the cross street. During the peak hour selected, the cross street admitted by signal 1,066 vehicles, of which 447 turned left, 447 went through on the service road, and 172 turned right. The leg was 3 lanes wide. At no time was there any serious backup although the left-turn lane did have about 20 loaded cycles (60-sec cycle).

The volume obtained during the one hour of peak flow (all of which was high-speed free flow) was: ramp, 1,092 vph, 1.1 percent trucks; lane 1, 868 vph, 2.4 percent trucks; diverge, 1,960 vph (ramp + lane 1); lane 2, 1,944 vph; lane 3, 1,988 vph; freeway, 4,800 vph (lane 1 + lane 2 + lane 3); total, 5,892 vph, 0.6 percent trucks freeway + ramp); approaching average per lane, 1,963 vph; going-away average per lane 1,599 vph (82 percent in lanes 2 and 3).

The highest 5-min count obtained was a 6,540-vph rate, which is an average of 2,180 vph per approaching lane. Despite the very high volumes, this location appeared capable of handling more traffic had the demand been present. The highest 5-min lane count was 184 vehicles. The highest 5-min diverging count (ramp + lane 1) was 183 vehicles.

Despite the high hour volume recorded, the study location at no time appeared in danger of queueing or developing a backup. A distinguishing feature of the entire operation was the steady demand. Ordinarily a location carrying 1,963 vph/lane average will have short periods of such high demand that the free-flow operation cannot be sustained. As previously mentioned, the highest 5-min count averaged only 2, 180 vph/ lane when expanded to one hour. The fact that the expressway traffic was outbound in the evening peak at an outlying location was also an advantage, as volumes downstream from the study location were less than 1, 600 vph/lane average.

The excellent performance obtained at this location suggests that perhaps attention is not being focused on the right aspects of exit ramps. The low commercial vehicle percentage, narrow angle of divergence (4°), adequate deceleration distance on the ramp, steady demand, limited weaving, and good target value of this location are all reasons for the high quality of performance. On the other hand, the deceleration lane is shorter than one might desire for a high-volume location yet it was used perfectly by the drivers. In summary, perhaps the need is simply for an exit, that can be seen and driven at normal lane 1 speeds. Other matters, such as commercial vehicles and traffic fluctuations, are not so easily controlled.