Development of Modified Procedures for Analysis of Ramp Capacity

ROGER P. ROESS

As part of an overall effort sponsored by the Federal Highway Administration to update and revise freeway capacity analysis procedures, Highway Capacity Manual procedures for ramps and ramp junctions were revised in order to (a) eliminate the dual procedure for differing levels of service; (b) eliminate cases in which on ramps are followed by off ramps and both are joined by an auxiliary lane, cases that are better treated as weaving sections; (c) adjust criteria to reflect passenger cars per hour rather than a vehicle population with 5 percent trucks; (d) update information on trucks in lane 1 of the freeway; (e) add material on left-handed ramps, ramps on 10-lane freeway segments, and ramps proper; and (f) add illustrative material on the impact of ramp geometry and acceleration-lane design. It is believed that the modifications recommended significantly simplify the use of, and eliminate many potential inconsistencies in, existing Highway Capacity Manual procedures.

There has been little in the way of new, basic research on the subject of ramp capacity by which to update procedures in the 1965 Highway Capacity Manual (1). Indeed, the only significant data base available for study is that collected by the then U.S. Bureau of Public Roads (BPR) and used in calibrating existing Highway Capacity Manual (HCM) procedures.

Nevertheless, in the course of an effort sponsored by the Federal Highway Administration (FHWA) to update and revise freeway-related elements of the HCM, it was considered necessary to modify existing procedures for freeway-ramp junctions to take into account the following important factors:

1. The format of the HCM ramp procedures is somewhat confusing because of the existence of different methods for levels of service A through C and levels of service D and E and because of the use of a large number of nomographs for various geometric configurations.

2. The use in the HCM of 5 percent trucks as a base vehicle population complicates computations and is inconsistent with other freeway-related parts of the manual.

3. HCM ramp procedures are affected by weaving-area procedures adopted elsewhere in the FHWA effort; the weaving procedures recommended are based on the National Cooperative Highway Research Program (NCHRP) method (2), which incorporates several geometric configurations now treated by using ramp techniques in the HCM.

4. Since the development of the 1965 HCM, some new material has been developed that permits treatment of cases and aspects not covered by HCM procedures.

In the light of these factors, HCM ramp procedures were examined for potential format modifications and simplifications and the addition of more recent material where it is available.

**BASIC PROCEDURE**

In considering a basic format for the presentation of ramp capacity procedures, three alternatives were examined:

1. The HCM procedure specified for levels of service A through C, based on a series of regression relations for various geometric configurations (18 relations, depicted in nomographs, were developed from the BPR data base referred to earlier).

2. The HCM procedure specified for level of service E (also used for level D), based on a limited data base collected in the state of California and often referred to as the California procedure or—in recognition of its developer, the late Karl Moskowitz—the Moskowitz procedure (the data base for this procedure is no longer extant);

3. The ramp procedure developed by Leisch in 1974 for FHWA (3) (based on the 1965 HCM, this procedure represents a radically different format, using multistep nomographs and a reduced number of different geometric cases).

The first major issue that must be examined is the need and justification for the dual procedures in the 1965 HCM for different levels of service. Examination of the BPR data base used for the HCM regression-based technique reveals that it contains data for all levels of service and is therefore valid throughout the full range of stable traffic flow. Furthermore, in the course of the Weaving Area Operations Study conducted at the Polytechnic Institute of New York for NCHRP, it was found that, throughout the full range of service levels, for the cases examined, the procedure for levels of service A through C was more accurate in the prediction of operating conditions than the procedure for levels of service E.

Thus it appeared that the only reasonable choice was between the HCM regression format and the Leisch format (3), which was based on the same data base. The California procedure was rejected as a basic technique but, because it is not configuration specific and can be applied to all cases, it was retained as a gross estimator for configurations and situations not covered by other methods.

The issue of whether to adopt the HCM format or the Leisch format was a difficult one. The Leisch format significantly reduced the number of different cases to be considered and might be thought to be simpler in application than the HCM format. On the other hand, the accuracy of the HCM relations was verified by the FHWA project team whereas information that would permit similar verification of the Leisch format was not available. In conducting comparative problem solutions, some degree of precision appeared to be lost by using the Leisch format. The FHWA project team therefore recommended the continued use of the HCM format (regression procedure) for ramp analysis, for the following reasons:

1. When problems are solved for level of service by using both the HCM and Leisch methods, results differ by one level in about 35 percent of the cases tried. Although these cases were most often borderline and the percentage difference in a numeric parameter, such as ramp volume (V), was generally less than 10 percent, this was considered a significant problem. Since the purpose of such procedures is often to find level of service, the loss of precision in the Leisch format, although not large in percentage terms, does lead to the step-function errors cited above.

2. The Leisch technique treats \( V_R \) as the
principal dependent variable. I, and other participants in the FHWA effort, felt that \( V_k \) is virtually always a demand value input into an analysis rather than the output of analysis. Philosophically, it was our view that, when a ramp design is considered, other general freeway features, ramp location, and demand volumes are known quantities. The most common use, therefore, is to solve for level of service to see if a given design will work. If not, the ramp location and/or design would have to be reconsidered. Only where ramp controls are being considered would \( V_k \) be the most probable output of computations.

3. In conversations with professionals in the field, opinion was mixed as to which format was preferable. Some supported the Leisch technique strongly, whereas others felt that it was somewhat complex. Overall, there was no strong preference among professionals for either technique.

4. The complexity of the HCM format is primarily the result of its presentation, which can be considerably simplified by eliminating ramp-weave cases and the duality of procedure for various service levels, as discussed previously.

5. The HCM methodology is a step-wise procedure that allows, indeed forces, the user to consider intermediate results and values. Members of the FHWA project team felt that this was essential to ensure the reasonableness of the results (it helps catch errors) and to give maximum insight into the analysis of field conditions.

The Leisch format would be useful as a computational aid if the computations could be simplified for the procedure adopted. However, the FHWA project team felt that, for the reasons stated above, it should not itself be adopted as a revised HCM technique.

PROCEDURAL MODIFICATIONS

The procedure for analysis of ramp-freeway junctions involves the determination of volumes in lane 1 of the freeway, just prior to a merge or diverge point. The regression-based procedure of the 1965 HCM contains 18 nomographs or equations to solve for this lane 1 volume (\( V_1 \)) for various ramp configurations. Of these, 5 can be eliminated, since they represent configurations that should be treated as weaving sections. The recommended procedure, therefore, retains 13 nomographs for the solution of \( V_1 \), as well as the California procedure for cases not covered by the nomographs.

The procedural steps and approach of the 1965 HCM (for levels of service A through C) have been retained. That procedure is used in the analysis mode in the following steps:

1. Establish all geometrics and demand volumes for the case to be considered, including upstream and downstream adjacent ramps and volumes.
2. Compute \( V_1 \) by using one of 13 nomographs, or estimate it by using the California procedure.
3. Find the percentage of trucks in lane 1 (a nomograph is provided).
4. Convert \( V_1 \), \( V_f \), and \( V_r \) to peak flow rates (passenger cars per hour) by using appropriate truck factors and the peak-hour factor (PIF).
5. Compute checkpoint volumes: \( V_m = V_1 + V_f \) and \( V_g = V_1 \) (\( V_g \) is taken immediately after merge and before diverge).
6. Compare \( V_f \), \( V_r \), and \( V_g \) with level-of-service criteria to determine level of service.

The modifications made to the 1965 HCM procedure in an attempt to reduce its complexity are discussed below.

**Nomographs**

A key element in the ramp-freeway-junction methodology is the determination of which nomograph to use to find \( V_f \). The HCM provides an index to aid in this selection. A similar index was prepared for the revised procedure. This index simplifies the HCM format by (a) eliminating five configurations now treated as weaving sections, (b) including several modifying notes in the HCM text on the index itself, (c) eliminating the ambiguity between the nomograph and California procedures, and (d) organizing the index according to "first ramp" and "second ramp" rather than on ramp and off ramp, as in the HCM, which creates confusion if both ramps are of the same type.

The nomographs themselves have been redrawn in a clarified format, and instructions for their use and special notations on modifications in special circumstances are highlighted. Figure 1 shows the new format developed. The sample nomograph applies to single-lane on ramps on six-lane freeways where there are adjacent upstream on ramps, with or without acceleration lanes. The normal range of the parameters is \( V_f = 1800-5400 \) vehicles/h, \( V_r = 100-1500 \) vehicles/h, \( V_u = 100-1400 \) vehicles/h, and \( D_u = 500-1500 \) ft. The steps in the solution are as follows:

1. Draw a line from the \( V_f \) value to the \( V_r \) value, intersecting turning line 1.
2. Draw a line from the \( V_u \) value to the \( D_u \) value, intersecting turning line 2.
3. Draw a line from the intersection on turning line 1 to the intersection on turning line 2, and read the solution on the \( V_1 \) line.

Equations for each of the nomographs referred to in Table 1 are given in Table 2, where

\[
V_1 = \text{lane 1 volume immediately upstream of an on ramp or off ramp,}
V_f = \text{total freeway volume immediately upstream of an on ramp or off ramp,}
V_r = \text{ramp volume for the ramp under consideration,}
D_u = \text{distance from ramp under consideration to adjacent upstream on ramp or off ramp,}
V_u = \text{ramp volume for adjacent on ramp or off ramp upstream of ramp under consideration,}
D_d = \text{distance from ramp under consideration to adjacent downstream on ramp or off ramp,}
V_c = \text{center-lane volume that divides at a major diverge point.}
\]

**Trucks in Lane 1**

Figure 8.22 of the HCM is used to compute the percentage of trucks in lane 1. Data available through the NCHRP Weaving Area Operations Study allowed for a partial recalibration of this figure, which was shown in that study to be grossly inaccurate for eight-lane freeways. The recalibrated curve is shown in Figure 2.

**Left-Hand Ramps and Ramps on 10-Lane Freeway Segments**

The 1965 HCM does not contain any material on left-hand ramps or ramps on 10-lane freeway segments. This is a serious lack, for the incidence...
Figure 1. Sample nomograph for determining lane 1 volume upstream of one-lane on ramps on six-lane freeways with upstream on ramps.

**Ramp Geometrics**

One of the difficulties of the HCM is that it does not account for the effect on performance of such variables as the length of the acceleration or deceleration lane, the angle of ramp convergence with the freeway, and relative grades. Little work has been done in this area, and incorporation of such variables into a working procedure is not yet possible.

Drew (5) has studied the effect on on-ramp merging performance of angle of convergence and acceleration-lane length. By using gap-acceptance models, Drew evaluated the impact of these variables on the percentage of gaps accepted by drivers, using a base, or "ideal," case of a 1200-ft acceleration lane and a 2° angle of convergence. Although interesting for its insights, however, Drew's work cannot be directly incorporated into a capacity analysis methodology because the ideal case adopted by Drew cannot be identified in existing data or procedures and Drew's use of the concept of capacity is not synonymous with the HCM's traditional use and implies, but does not define, a measure of service quality.

Table 3 was developed from Drew's work to show the potential impact of angle of convergence and acceleration-lane length on performance. It is not intended for use as a computational device.

The table does, however, indicate the extreme importance of these factors to ramp operation. The fact that these are not included in current procedures may be a serious deficiency that should be addressed through basic research.

**Levels of Service for Ramp-Freeway Juncions**

Table 8.1 of the HCM gives criteria for ramp-junction level of service in terms of limiting values of $V_m$ (merge volume), $V_d$ (diverge volume), and $V_w$ (weaving volume per 500 ft of distance). Operating criteria associated with these conditions are vague and are not clearly defined in terms of speeds or other operating parameters. It is implied that these limiting volumes are such that the indicated level of service would prevail on the freeway as a whole, as defined in HCM Table 9.1, for the condition described.

This leads to an immediate problem in the current context. As part of the FHWA effort, Table 9.1 of the HCM has been modified (6). This would suggest that corresponding values of $V_m$, $V_d$, and $V_w$ in HCM Table 8.1 should also be recalibrated. Unfortunately, there is no sound basis or data that would make it possible to accomplish this.

Table 4 compares HCM Table 9.1 service volumes for freeways (expressed on a per-lane basis) and HCM Table 8.1 criteria for $V_m$, $V_d$, and $V_w$. This comparison reflects the view of HCM developers that point flows (merge, diverge, and weave) could be higher than single-lane service volumes for a given level of service because of the restrictive area of their influence.

Subsequent research has shown that the extent of the influence of "point" flows can be quite extended, often as much as 5000-6000 ft of highway at poorer levels of service (7). In view of this, it was considered unwise to continue this policy. The criteria recommended for $V_d$, $V_m$, and $V_w$ are given in Table 5 and are compared with the criteria recommended for normal freeway conditions as a result of modifying HCM Table 9.1. The selection of these criteria was somewhat judgmental and was based on the following considerations:
1. It was felt that neither merge-, diverge-, nor weaving-volume criteria for a given level of service should be higher than the average per-lane service volume for that level on basic freeway sections. Given the additional turbulence involved in these maneuvers, and given that the criteria should be such that the freeway as a whole operates at the stated level, $V_m$, $V_d$, and $V_w$ criteria should be somewhat lower than corresponding criteria for $V_f$.

2. In general, a diverge movement is less disruptive to flow than a merge. Therefore, merge-volume criteria should be more restrictive than diverge criteria for any given level of service.
3. The weaving criterion in HCM Table 9.1 appears to be entirely too high. Although this criterion is only used in cases of an on ramp followed by an off ramp without an auxiliary lane, it should be somewhat in line with weaving volumes predicted by

the weaving methodology developed for the FHWA effort (4), tempered by the fact that in such sections weaving is really a merge followed by a diverge movement.

4. Criteria should be expressed in passenger cars per hour for consistency with other freeway procedures. This is not a major problem, since the nomographs for solution of \( V_1 \) were calibrated and are solved by using mixed vehicles per hour with whatever percentage of trucks exists. Conversions to a base population (5 percent trucks in the HCM) are required only to compare with level-of-service criteria. Thus, the nomograph relations need not be adjusted—only the level-of-service criteria, to reflect passenger cars per hour.

5. For consistency with other material developed for the FHWA, criteria represent peak flow rates (for PHF = 1.00) rather than full-hour volumes.

Clearly, the subject of criteria for ramp level of service is one that should be carefully studied in the future. The recommendations given here are reasonable and consistent. In view of the lack of hard data for analysis, they result from the only

<table>
<thead>
<tr>
<th>Nomograph</th>
<th>Equation</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>A4.1</td>
<td>( V_1 = 136 + 0.345V_r - 0.115V_t )</td>
<td>Not for use if upstream adjacent on ramp exists within 200 ft</td>
</tr>
<tr>
<td>A4.2</td>
<td>( V_1 = 165 + 0.345V_r + 0.52V_t )</td>
<td>Not for use if upstream adjacent on ramp exists within 3200 ft</td>
</tr>
<tr>
<td>A4.3</td>
<td>( V_1 = 202 + 0.36V_r - 0.496V_t - 0.069V_u + 0.096V_f )</td>
<td>For use only if upstream adjacent on ramp exists within 3200 ft</td>
</tr>
<tr>
<td>A4.4</td>
<td>( V_1 = 166 + 0.28V_r ) (for ( V_r &lt; 600 ) vehicles/h)</td>
<td>For use with loop ramps only</td>
</tr>
<tr>
<td>A4.5</td>
<td>( V_1 = 123 + 0.376V_r - 0.142V_t )</td>
<td>For use only if upstream adjacent on ramp exists within 2000 ft</td>
</tr>
<tr>
<td>A4.6</td>
<td>( V_1 = -121 + 0.244V_r - 0.085V_u + 640V_u/D_c )</td>
<td>( V_u ) refers to an upstream off ramp within 2600 ft; if none exists, set ( V_u = 50 )</td>
</tr>
<tr>
<td>A4.7</td>
<td>( V_1 = 94 + 0.231V_r + 0.473V_t + 214V_u/D_c )</td>
<td>( V_u/D_c ) refers to a downstream off ramp within 5700 ft; if none exists, set ( 640V_u/D_c = 5 )</td>
</tr>
<tr>
<td>A4.8</td>
<td>( V_1 = 574 + 0.228V_r - 0.194V_t + 0.714V_u + 0.274V_f )</td>
<td>For use only if upstream on ramp exists within 1400 ft</td>
</tr>
<tr>
<td>A4.9</td>
<td>( V_1 = 312 + 0.201V_r + 0.111V_t )</td>
<td>For use only if there is no adjacent downstream off ramp within 3000 ft</td>
</tr>
<tr>
<td>A4.10</td>
<td>( V_1 = -353 + 0.199V_r - 0.057V_t - 0.486V_u )</td>
<td>For use if downstream adjacent off ramp exists within 1500-3000 ft</td>
</tr>
<tr>
<td>A4.11</td>
<td>( V_1 = 54 + 0.070V_r + 0.049V_t )</td>
<td>Special case (4)</td>
</tr>
<tr>
<td>A4.12</td>
<td>( V_1 = 64 + 0.285V_r + 0.041V_t )</td>
<td>( V_u ) is center-lane volume on six-lane freeway just prior to major diverge into two four-lane freeways</td>
</tr>
</tbody>
</table>

Table 3. Effect of ramp geometrics on gaps accepted by merging vehicles.

<table>
<thead>
<tr>
<th>Angle of Convergence (°)</th>
<th>Percentage of Ideal Case by Length of Acceleration Lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>1200 ft</td>
<td>2000 ft</td>
</tr>
<tr>
<td>1000 ft</td>
<td>2000 ft</td>
</tr>
<tr>
<td>800 ft</td>
<td>2000 ft</td>
</tr>
<tr>
<td>600 ft</td>
<td>2000 ft</td>
</tr>
<tr>
<td>400 ft</td>
<td>2000 ft</td>
</tr>
</tbody>
</table>

Table 4. Relation between freeway and ramp level of service in 1965 HCM.

<table>
<thead>
<tr>
<th>Level of Service</th>
<th>( V_f ) (passenger cars/h)</th>
<th>( V_m ) (vehicles/h)</th>
<th>( V_c ) (passenger cars/h)</th>
<th>( V_w ) (vehicles/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>800</td>
<td>1000</td>
<td>1100</td>
<td>1100</td>
</tr>
<tr>
<td>B</td>
<td>1100</td>
<td>1200</td>
<td>1300</td>
<td>1300</td>
</tr>
<tr>
<td>C</td>
<td>1600</td>
<td>1700</td>
<td>1800</td>
<td>1800</td>
</tr>
<tr>
<td>D</td>
<td>1800</td>
<td>1900</td>
<td>2000</td>
<td>2000</td>
</tr>
<tr>
<td>E</td>
<td>2000</td>
<td>2000</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>F</td>
<td>--</td>
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</tr>
</tbody>
</table>

Table 5. Relation between freeway and ramp level-of-service criteria recommended in FHWA study.

<table>
<thead>
<tr>
<th>Level of Service</th>
<th>( V_f ) (passenger cars/h)</th>
<th>( V_m ) (passenger cars/h)</th>
<th>( V_c ) (passenger cars/h)</th>
<th>( V_w ) (passenger cars/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>800</td>
<td>750</td>
<td>800</td>
<td>500</td>
</tr>
<tr>
<td>B</td>
<td>1300</td>
<td>1200</td>
<td>1300</td>
<td>700</td>
</tr>
<tr>
<td>C</td>
<td>1700</td>
<td>1550</td>
<td>1650</td>
<td>1300</td>
</tr>
<tr>
<td>D</td>
<td>1925</td>
<td>1800</td>
<td>1900</td>
<td>1550</td>
</tr>
</tbody>
</table>

For PHF = 1.00.
For six-lane freeways, 70-mile/h average highway speed.
congestion through the application of systems and capacity figures by (a) assuming that the percentage of a ramp based on its design speed. These values a Level of service not achievable because of restricted design speed.

RAMP METERING

Although much study has been devoted to the control methodologies of freeway surveillance and control. Level-of-service criteria were developed from various sources. Since many urban areas attempt to deal with problems of freeway congestion through the application of systems and methodologies of freeway surveillance and control. Although much study has been devoted to the control aspects of these systems, virtually no work has been done on their capacity implications.

DEVELOPMENT OF CRITERIA FOR RAMPS PROPER

The 1965 HCM treats only the capacity analysis of ramp-freeway terminals. Material from the American Association of State Highway Officials "Blue Book" (8) was adapted by Leisch (3) to yield the capacity of a ramp based on its design speed. These values were used in this work and were further modified to provide approximate level-of-service guidelines. Level-of-service criteria were developed from capacity figures by (a) assuming that the percentage of capacity for a given ramp level of service should be similar to the percentage of capacity for the same level on basic freeway sections. A special procedure for computing , as the dependent variable in an analysis is also given. On the subject of ramp metering, it is reasonable to ask, What limiting value of can be permitted to enter the freeway without causing level of service to be poorer than a given level? The answer can be found by (a) assuming a value for as the dependent variable in an analysis is also given. On the subject of ramp metering, it is reasonable to ask, What limiting value of can be permitted to enter the freeway without causing level of service to be poorer than a given level? The answer can be found by (a) assuming a value for as the dependent variable in an analysis is also given.

SAMPLE PROBLEMS

The simple problems discussed below illustrate the use of some of the revised procedures discussed in this paper.

Analysis of Ramp-Freeway Junctions

To determine the expected level of service provided at the two ramp-freeway junctions shown in Figure 3, follow the steps given below:

1. Establish all geometrics and demand volumes. This has been done in the problem statement.

2. Compute . Table 1 indicates that Figure A4.1 should be used to compute for the first on ramp and Figure A4.5 for the second (the equations for these figures are found in Table 2):

\[
\begin{align*}
V_{fA} & = 136 + 0.345 V_{fa} - 0.115 V_{ra} \\
V_{iA} & = 136 + 0.345 (2000) - 0.115 (300) \\
V_{iA} & = 136 + 690 - 35 = 791 \text{ passenger cars/h.} \\
V_{fB} & = 123 + 0.376 V_{fb} - 0.142 V_{rb} \\
V_{iB} & = 123 + 0.376 (2300) - 0.142 (500). \\
V_{iB} & = 123 + 865 - 71 = 917 \text{ passenger cars/h.} \\
V_{rb} & = 0.87 \\
V_{fa} & = 2000/0.87 = 2299, \\
V_{fb} & = 2300/0.87 = 2644. \\
V_{ra} & = 300/0.87 = 345, \\
V_{rb} & = 500/0.87 = 575, \\
V_{ia} & = 791/0.87 = 909, \text{ and } V_{ib} = 917/0.87 = 1054.
\end{align*}
\]

2. Because ramp metering not only limits total demand but also smooths that demand by preventing simultaneous multivehicle entries, it can potentially alter the basic nature and characteristics of merging maneuvers and thus alter basic capacities and/or service volumes.

In the work discussed here, the first position was adopted merely because there are no available data or research on which to base an evaluation of the second position. The FHWA project team believes that some original research will be required to resolve the issue.

Because of the importance of ramp metering, material was developed that treats the subject qualitatively. A special procedure for computing as the dependent variable in an analysis is also given. On the subject of ramp metering, it is reasonable to ask, What limiting value of can be permitted to enter the freeway without causing level of service to be poorer than a given level? The answer can be found by (a) assuming a value for , (b) computing by using nomographs or California-procedure approximation, (c) finding the limiting value of for the level of service of interest, and (d) computing . The procedure is iterated until the assumed is reasonably equivalent to the computed. In summary, because no substantial new work on the subject of ramps has taken place since the publication of the 1965 HCM, the procedures developed for ramps as part of the FHWA project on freeway capacity-analysis procedures were based on the results of simplifications and special-case extensions of the HCM. Ramp capacity, therefore, is a subject that will require substantial research if significant improvements on current techniques are to be expected for the mid-1980s edition of the HCM.
5. Compute checkpoint volumes and compare with criteria in Table 5:

<table>
<thead>
<tr>
<th>Checkpoint Volume</th>
<th>Level of Service</th>
</tr>
</thead>
<tbody>
<tr>
<td>V_{mA} = 345 + 909 = 1254 passenger cars/h</td>
<td>C</td>
</tr>
<tr>
<td>V_{mB} = 575 + 1054 = 1629 passenger cars/h</td>
<td>D</td>
</tr>
<tr>
<td>V_{fA} = 2299/2 = 1150 passenger cars/h/l</td>
<td>B</td>
</tr>
<tr>
<td>V_{fB} = 2644/2 = 1322 passenger cars/h/l</td>
<td>C</td>
</tr>
</tbody>
</table>

Clearly, the second merge is the critical operating element that causes the overall performance level to be level of service D.

Analysis of Ramp Metering

It is desired that V_r be controlled by establishing a maximum flow rate through ramp metering at the location shown in Figure 4. If a fixed-time ramp meter is used, at what rate should ramp vehicles be allowed to enter the traffic stream if the level of service is not to be permitted to be worse than C?

The question asks for a solution of a maximum value of V_r so that the level of service is C. The trial-and-error method described in the previous discussion of ramp metering is used.

From Table 5, the maximum merge service volume for level of service C is 1550 passenger cars/h (peak flow rate). For a PHF of 0.90, this is equivalent to an hourly volume of 1550 x 0.90 = 1395 passenger cars/h. Considering the situation given in the problem, a tabular computation can be set up as follows [V_{i} is computed from Figure A4.1 (4), and the formula for the computed V_r is 1395 - V_{i}]:

<table>
<thead>
<tr>
<th>Assumed</th>
<th>Computed</th>
</tr>
</thead>
<tbody>
<tr>
<td>V_0</td>
<td>V_1</td>
</tr>
<tr>
<td>200</td>
<td>810</td>
</tr>
<tr>
<td>400</td>
<td>775</td>
</tr>
<tr>
<td>600</td>
<td>760</td>
</tr>
<tr>
<td>650</td>
<td>750</td>
</tr>
</tbody>
</table>

A metering rate of 650 passenger cars/h, or one car every 3600/650 = 5.54 s =-say, 5.5 s—would be set.

A more precise solution can be found by using the equation for Figure A4.1 directly:

\[ V_{r} = 136 + 0.345 V_{f} - 0.115 V_{i} \]  (I)

and considering that \( V_{f} = 1395 - V_{i} \). Substituting for \( V_{i} \) in the \( V_{r} \) equation,

\[ V_{r} = 1395 - (136 + 0.345 V_{f} - 0.115 V_{r}) \]
\[ V_{r} = 1259 - 0.345 V_{f} + 0.115 V_{r} \]
\[ 0.885 V_{r} = 1259 - 0.345 V_{f} \]
\[ V_{r} = (1259 - 0.345(2000))/0.885 = 643 \text{ passenger cars/h} \]

CONCLUSIONS

It is believed that the modifications to HCM procedures for analysis of ramp capacity reported in this paper both simplify and clarify the application of those procedures. It is hoped, however, that new, basic research in the area of ramp capacity will be possible before the publication of the new HCM in the mid-1980s.

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The views and opinions expressed here are mine. The procedures described do not represent standards or methods endorsed by FHWA or any other agency.

REFERENCES