Development of Modified Procedures for Analysis of Ramp Capacity

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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); and
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 level 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_R), 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_R 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_R 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 (PHF).
5. Compute checkpoint volumes: $V_m = V_1 + V_r$ and $V_d = V_1$ (V_f is taken immediately after merge and before diverge).
6. Compare V_f, V_m, and V_d 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_1. 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 = lane 1 volume immediately upstream of an on ramp or off ramp,
V_f = total freeway volume immediately upstream of an on ramp or off ramp,
V_r = ramp volume for the ramp under consideration,
D_u = distance from ramp under consideration to adjacent upstream on ramp or off ramp (ft),
V_u = ramp volume for adjacent on ramp or off ramp upstream of ramp under consideration,
V_d = ramp volume for adjacent on ramp or off ramp downstream of ramp under consideration,
D_d = distance from ramp under consideration to adjacent downstream on ramp or off ramp (ft), and
V_c = center-lane volume that divides at a major diverge point.

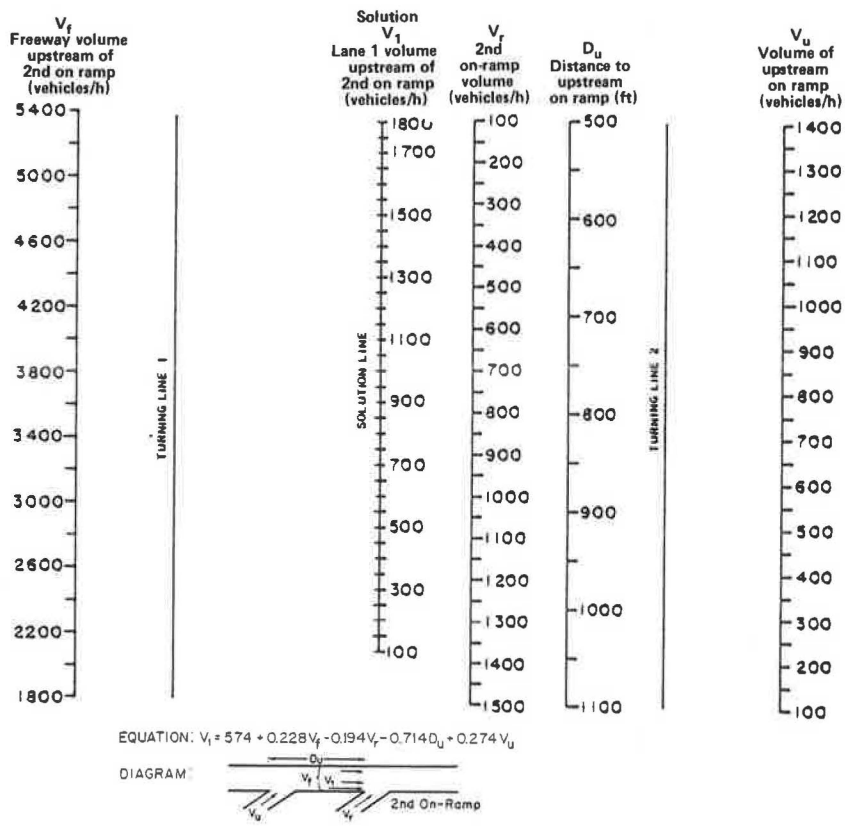
Trucks in Lane 1

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.



of both has increased since 1965. No new data are available for such situations, but Leisch (3) developed approximation procedures for both cases, and these were adopted "as is" for the current work.

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 JUNCTIONS

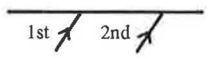
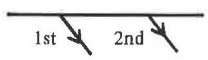
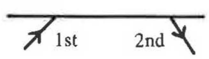
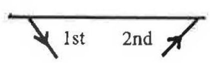
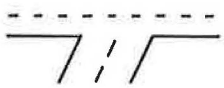
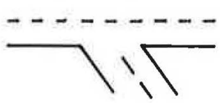
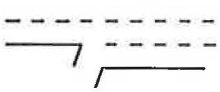
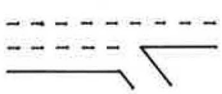
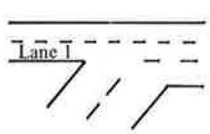
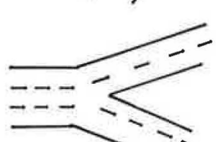
Table 8.1 of the HCM gives criteria for ramp-junction level of service in terms of limiting values of V_m (merge volume), V_d (diverge volume), and V_w (weaving volume per 500 ft of distance). Operating criteria associated with these conditions are vague and are not clearly defined in terms of speeds 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:

Table 1. Index to nomographs and procedures for analysis of ramp terminals.

		Four-Lane Freeway		Six-Lane Freeway		Eight-Lane Freeway	
Configuration	Diagram	First Ramp	Second Ramp	First Ramp	Second Ramp	First Ramp	Second Ramp
Isolated one-lane on ramp		Figure A4.1	–	Figure A4.6	–	Figure A4.9	–
Isolated one-lane off ramp		Figure A4.2	–	Figure A4.7	–	Approximate by using Table 4.3 and Figure 4.3	
Adjacent one-lane on ramps		Figure A4.1	Figure A4.5	Figure A4.6	Figure A4.8	Approximate by using Table 4.3 and Figure 4.3	Approximate by using Table 4.3 and Figure 4.3
Adjacent one-lane off ramps		Figure A4.2[a]	Figure A4.2	Figure A4.7[a]	Figure A4.7	Approximate by using Table 4.3 and Figure 4.3	Approximate by using Table 4.3 and Figure 4.3
On ramp followed by off ramp		Figure A4.1	Figure A4.3	Figure A4.6	Figure A4.7	Figure A4.10	Approximate by using Table 4.3 and Figure 4.3
Off ramp followed by on ramp		Treat as isolated ramps			Figure A4.6	Treat as isolated ramps	
Loop ramps		Figure A4.4	Figure A4.3	Figure A4.6	Figure A4.7	Figure A4.10	Approximate by using Table 4.3 and Figure 4.3
Two-lane on ramps		NA	–	Figure A4.11		NA	–
Two-lane off ramps		–	NA	–	Figure A4.12	–	NA
Addition of lane at on ramp		Merge criteria in Table 4.1 may be applied directly to on-ramp volume as a checkpoint					
Dropping of lane at off ramp		Diverge criteria in Table 4.1 may be applied directly to off-ramp volume as a checkpoint					
Major junctions		Assume that ramp lane B carries an amount of traffic equal to merge checkpoint volume in Table 4.1 for assumed level of service; ramp lane A then carries remaining ramp traffic; compute lane 1 volume by using Figure A4.1 (four-lane freeway), A4.6 (six-lane freeway), or A4.9 (eight-lane freeway), entering with V_r = ramp lane A volume; find checkpoint levels of service; continue computations until assumed level agrees with results					
Major forks (diverges)		NA	–	Figure A4.13	–	NA	–

[a]Use this figure to find V_1 in advance of the first ramp, but enter with a V_r equal to the total volume on both off ramps. This technique is valid where the distance between ramps is less than 800 ft. Where the distance between ramps is between 800 and 4000 ft, use Table 4.3 and Figure 4.3 to approximate the situation. Where the distance between ramps is greater than 4000 ft, the ramps are treated as if they were independent (isolated).

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 (or 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 2. Equations for nomographs given in Table 1.

Nomograph	Equation	Notes
A4.1	$V_1 = 136 + 0.345V_f - 0.115V_r$	Not for use if upstream adjacent on ramp exists within 200 ft
A4.2	$V_1 = 165 + 0.345V_f + 0.520V_r$	Not for use if upstream adjacent on ramp exists within 3200 ft
A4.3	$V_1 = 202 + 0.362V_f + 0.496V_r - 0.069D_u + 0.096V_u$	For use only if upstream adjacent on ramp exists within 3200 ft
A4.4	$V_1 = 166 + 0.280V_f$ (for $V_r < 600$ vehicles/h) $V_1 = 128 + 0.482V_f - 0.301V$ (for V_r between 600–1200 vehicles/h)	For use with loop ramps only
A4.5	$V_1 = 123 + 0.376V_f - 0.142V_r$	For use only if upstream adjacent on ramp exists within 2000 ft
A4.6	$V_1 = -121 + 0.244V_f - 0.085V_u + 640\ V_d/D_d$	V_u refers to an upstream off ramp within 2600 ft; if none exists, set $V_u = 50$ V_d/D_d refers to a downstream off ramp within 5700 ft; if none exists, set $640\ V_d/D_d = 5$
A4.7	$V_1 = 94 + 0.231V_f + 0.473V_r + 214\ V_u/D_u$	V_u/D_u refers to an upstream on ramp within 5700 ft; if none exists set $215\ V_u/D_u = 2$
A4.8	$V_1 = 574 + 0.228V_f - 0.194V_r - 0.714D_u + 0.274V_u$	For use only if upstream on ramp exists within 1400 ft
A4.9	$V_1 = -312 + 0.201V_f + 0.117V_r$	For use only if there is no adjacent downstream off ramp within 3000 ft
A4.10	$V_1 = -353 + 0.199V_f - 0.057V_r - 0.486V_d$	For use if downstream adjacent off ramp exists within 1500-3000 ft
A4.11	$V_1 = 54 + 0.070V_f + 0.049V_r$	—
A4.12	—	Special case (4)
A4.13	$V_c = 64 + 0.285V_f + 0.041V_r$	V_c is center-lane volume on six-lane freeway just prior to major diverge into two four-lane freeways

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

Angle of Convergence (°)	Percentage of Ideal Case by Length of Acceleration Lane: 1200 ft	1000 ft	800 ft	600 ft	400 ft
2	100.0	96.8	90.3	64.5	32.3
4	80.6	77.4	48.4	32.3	17.7
6	45.2	45.2	32.3	24.2	11.3
8	33.8	33.8	25.8	15.5	9.7
10	32.3	32.3	24.5	13.5	8.1

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

Level of Service	Maximum Allowable Value[a]: V_f (passenger cars/h/lane)[b,c]	V_m (vehicles/h)[d]	V_d (vehicles/h)[d]	V_w (vehicles/h)[d]
A	800	1000	1100	800
B	1167	1200	1300	1000
C	1600	1700	1800	1450
D	1800	1800	1900	1800
E	2000	2000	2000	2000
F	—	—	—	—

[a]For PHF = 1.00.
[b]For six-lane freeways, 70-mile/h average highway speed.
[c]HCM Table 9.1.
[d]HCM Table 8.1.

Figure 2. Percentage of trucks in lane 1.

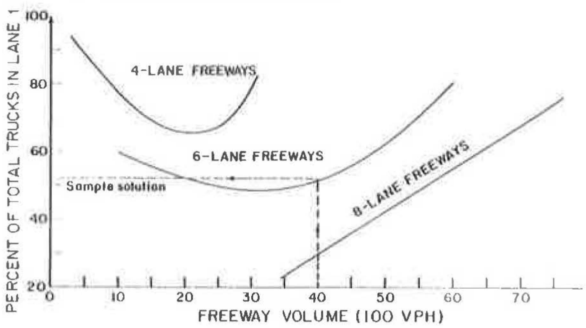


Table 5. Relation between freeway and ramp level-of-service criteria recommended in FHWA study.

Level of Service	Maximum Allowable Value[a]: V_f (passenger cars/h/lane)[b]	V_m (passenger cars/h)	V_d (passenger cars/h)	V_w (passenger cars/h)
A	800	750	800	500
B	1300	1200	1300	700
C	1700	1550	1650	1300
D	1925	1800	1900	1550
E	2000	2000	2000	2000
F	—	—	—	—

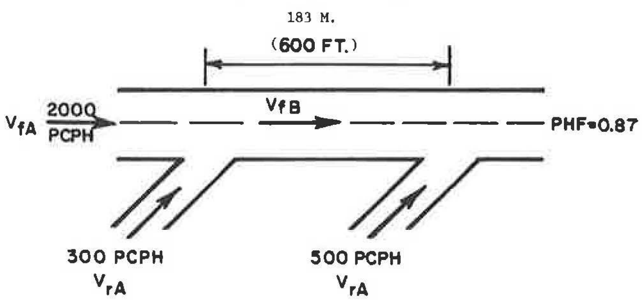
[a]For PHF = 1.00.
[b]For six-lane freeways, 70-mile/h average highway speed.

Table 6. Approximate service volumes for single-lane ramps.

Ramp Design Speed (miles/h)	Service Volume by Level of Service (passenger cars/h)				
	A	B	C	D	E
<20	–[a]	–[a]	–[a]	–[a]	1250
20-29	–[a]	–[a]	–[a]	1025	1450
30-39	–[a]	–[a]	1125	1200	1600
40-49	–[a]	1000	1250	1325	1650
≥50	700	1050	1300	1500	1700

Note: Level of service F varies widely.
For two-lane ramps, multiply the given values by 1.7 for ≤ 20 miles/h, by 1.8 for 20-29 miles/h, by 1.9 for 30-49 miles/h, and by 2.0 for ≥50 miles/h.
[a]Level of service not achievable because of restricted design speed.

Figure 3. Illustration for ramp-freeway-terminal problem.



type of approach that can currently be taken.

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 and (b) not allowing for better levels of service on ramps that have a restricted design speed, a safety consideration modeled after similar use in basic freeway criteria, where better levels of service are not allowed for highways with a reduced average highway speed.

The criteria developed are given in Table 6.

RAMP METERING

The subject of ramp metering has become one of increasing importance in recent years, 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 (9), virtually no work has been done on their capacity implications.

Two positions can be taken regarding the capacity implications of ramp metering:

1. Ramp metering has no impact on capacity or service volume. It serves merely to limit demand so that breakdown does not occur and/or certain prescribed levels of service do not deteriorate.

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 V_r 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 V_r 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 V_r, (b) computing V_1 (by using nomographs or California-procedure approximation), (c) finding the limiting value of V_m for the level of service of interest, and (d) computing $V_r = V_m - V_1$. The procedure is iterated until the V_r assumed is reasonably equivalent to the V_r 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 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.

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.

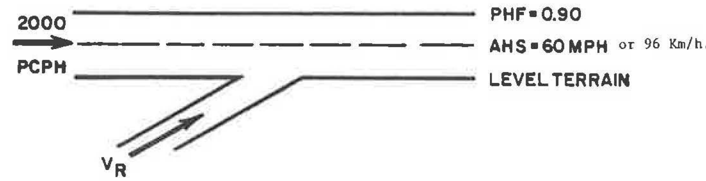
2. Compute V_1. Table 1 indicates that Figure A4.1 should be used to compute V_1 for the first on ramp and Figure A4.5 for the second (the equations for these figures are found in Table 2):

$V_{1A} = 136 + 0.345\ V_{fA} - 0.115\ V_{rA}$.
$V_{1A} = 136 + 0.345\ (2000) - 0.115\ (300)$.
$V_{1A} = 136 + 690 - 35 = 791$ passenger cars/h.
$V_{1B} = 123 + 0.376\ V_{fB} - 0.142\ V_{rB}$.
$V_{1B} = 123 + 0.376\ (2300) - 0.142\ (500)$.
$V_{1B} = 123 + 865 - 71 = 917$ passenger cars/h.

3. Find the percentage of trucks in lane 1 (since all volumes are given in passenger cars per hour, this step is not necessary).

4. Convert volumes to peak flow rates in passenger cars per hour. Since volumes are already given in these terms, peak flow rates in passenger cars per hour are computed by dividing by the PHF (0.87 in this case): $V_{fA} = 2000/0.87 = 2299$, $V_{fB} = 2300/0.87 = 2644$, $V_{rA} = 300/0.87 = 345$, $V_{rB} = 500/0.87 = 575$, $V_{1A} = 791/0.87 = 909$, and $V_{1B} = 917/0.87 = 1054$.

Figure 4. Illustration for ramp-metering problem.



5. Compute checkpoint volumes and compare with criteria in Table 5:

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

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_1 is computed from Figure A4.1 (4), and the formula for the computed V_r is 1395 - V_1]:

Assumed V_θ	V_1	Computed V_θ
200	810	585
400	775	620
600	760	635
650	750	645

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:

$$V_1 = 136 + 0.345\ V_f - 0.115\ V_r \quad (1)$$

and considering that V_r = 1395 - V_1. Substituting for V_1 in the V_r equation,

$V_r = 1395 - (136 + 0.345\ V_f - 0.115\ V_r)$.
$V_r = 1259 - 0.345\ V_f + 0.115\ V_f$.
$0.885\ V_r = 1259 - 0.345\ V_f$.
$V_r = (1259 - 0.345\ V_f)/0.885$.
$V_r = [1259 - 0.345(2000)]/0.885 = 643$ 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.

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